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FATIGUE STRENGTH OF TWO STEEL DETAILS

by

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ABSTRACT

The purpose of this investigation is twofold. In Part A the fatigue behavior of loose riveted connections, actual riveted members taken from service, and the effect of structural rehabilitation obtained by replacing rivets with high-strength bolts are examined. In Part B the fatigue behavior of groove welds with backing bars, the entire detail being perpendicular to the direction of applied stress, is examined.

Riveted connections are not classified for fatigue in either the Canadian Standards Association Standard S16.1-M78, Steel Structures for Buildings-Limit States Design or Canadian Standards Association Standard S6-1974, Design of Highway Bridges. However, in the American Association of State Highway and Transportation Officials Standard Specifications for Highway Bridges, 1977, and in the American Railway Engineering Association Manual for Railway Engineering, 1979, this detail is placed into Category D. The results of tests on three beams with holes (representing loose riveted connections), three beams with bolts filling the holes, and four riveted members are presented in Part A. The study confirms the use of Category D for riveted connections provided the rivets are not loose. If riveted connections are suspected of being loose it is suggested that Category E be used for fatigue design. The study also showed that an effective way of structural rehabilitation for existing riveted structures is to replace

the rivets with high-strength bolts.

Few guidelines exist for the fatigue design of a groove weld with backing bar located perpendicularly to the direction of stress. Neither the CSA Standard S16.1-M78 nor the American Institute of Steel Construction Specification, 1978 permit the use of this detail. The British Standard BS 5400, Part 10, 1980, classifies this detail as category F or G depending on the location of the fillet welds with respect to the plate edge. The basis for these choices is predominently theoretical. In Part B the findings of twelve tests on groove welds with the backing bar(attached by intermittent fillet welds) located perpendicularly to the applied stress are reported. The data are compared with analytical results. The study indicates that this detail should be designated as Category C.

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1. INTRODUCTION

1.1. General

Fatigue failure is the result of repeated applications of stress causing microcracks which can propagate so as to bring about a failure of the remaining cross section. The stresses producing these fatigue fractures can be lower in magnitude than those causing yielding of the uncracked section. Three principle factors affect the fatigue strength of steel structural members: stress range; number of load applications; and the type of structural detail (1).

From knowledge accumulated in recent years a satisfactory basis for fatigue design of structural steel members containing welded details has been established. Although not nearly as much attention has been directed towards members with high strength bolted connections, re-evaluation of earlier data has produced an apparently adequate foundation for design. Although there are still many areas where additional information is required, the designer generally has a satisfactory basis for the fatigue design of steel members that contain welded or bolted details.

1.2. Statements of the Problems

In the past, bridge structures and material handling systems were erected using riveted construction. However, since this form of construction is now virtually obsolete,

the fatigue strength of riveted details has not been extensively evaluated in terms of the modern approach. Many riveted structures have now reached an age when renovation or replacement must be considered. One important factor in evaluating these structures is the fatigue life of the riveted connection. Part A of this investigation examines this detail. A natural extension of this was to examine the effects of structural rehabilitation of riveted joints when rivets are replaced with high-strength bolts.

In welded construction of steel structures, groove welding from one side is often convenient and, in some cases, essential because of limited access. In these cases, the groove weld is made using a temporary or permanent backing bar. Prior to making the groove weld, the backing bar is usually attached to the plates that are to be spliced by short, intermittent fillet welds, generally arranged in a staggered fashion.

The backing bar is usually left in place in situations where the structure and the weld are not fatigue loaded. It is economically more desirable and yet not detrimental to the joint strength. However, this situation does not necessarily apply when fatigue is a factor. In situations where fatigue is a consideration the CSA Standard W59-1977, Welded Steel Construction(Metal Arc Welding) (2) allows the backing bar to remain when the groove weld with backing bar is located parallel to the applied stress. The backing bar is not allowed to remain when the groove weld is situated

perpendicular to the applied stress. Therefore, even for a low stress range the designer does not have the option of using this detail transversely to the direction of stress without removal of the backing bar. The economic penalty of doing this may be significant. Furthermore, mechanical removal of the backing bar will not necessarily eliminate the metallurgical changes or cracks which may have been introduced. In Part B of this investigation data concerning fatigue strength of this detail, when it is located perpendicularly to the direction of applied stress, has been obtained through testing. A finite element analysis has been used to assist in the interpretation of the test results.

1.3. Objectives

Part A - Riveted Connections

The objectives of Part A are:

1. To investigate the fatigue strength of joints with completely loose rivets.
2. To investigate the benefits of replacing rivets with high-strength bolts in critical locations of connections.
3. To consider the effect of using bolts rather than welds to fill in improperly located holes.
4. To investigate the fatigue strength of riveted connections taken from service.

Part B - Groove Welds with Backing Bars

The objectives of Part B are:

1. To investigate the fatigue strength of groove welds with backing bars located transversely to the direction of stress.
2. To recommend code provisions for this detail.
3. To make recommendations for further testing.

PART A - RIVETED CONNECTIONS

2. LITERATURE SURVEY

2.1. Previous Testing of Riveted Connections

In the 1850's machinery was being designed that made more demanding use of rivets. The developers were concerned with perfecting the principle mechanics of the machinery. Little attention was given to the riveted connections. After numerous failures the first known investigation of static and fatigue behavior in riveted joints was carried out (3).

Since then a number of fatigue tests have been performed in North America (4,5,6,7,8). These studies examined the effect of the rivet clamping force, the rivet grip, the influence of the bearing ratio, and other variables. Figure 2.1 presents a plot of the test results from all sources available up to 1976 (9). The great majority of these tests were made using new specimens.

The substantial scatter in the test data has been attributed mainly to the influence of the clamping force. The presence of a clamping force in a riveted connection can be explained by the procedure used in hot-driving a rivet. The rivet is heated and inserted in the hole. Pressure is then applied to the preformed head while at the same time the plain end of the rivet is squeezed to form a rounded head. Upon cooling, the shank of the rivet shrinks providing

a clamping force. A paper by Wyly and Scott cites a study by Yoshima and McCammon in which the clamping force of short 3/4 in. diameter rivets in double lap joints was examined (7). The clamping force was found to vary considerably.

A study by Munse, Wright and Newmark found that the fatigue strength of bolted specimens, which were identical except for variations in clamping force, increased as the fastener clamping force increased (10). The greater clamping forces allow the transfer of load by friction between the plates. Bearing stresses are reduced, resulting in less severe stress concentrations near the hole. An Illinois study tested several apparently identical riveted specimens (7). After several tests, a considerable scatter in the fatigue results was noticed. It appeared that variability in the normal clamping of the rivets was responsible for the scatter. Reduced clamping was, therefore, introduced in the testing of some of the remaining specimens. Reduced clamping in the rivets was accomplished by machining away most of the rivet heads or by pressing the rivet head in order to thrust the shank down slightly. The results of these tests showed much less variation. These studies substantiate the view that clamping force has a significant effect on the fatigue life of riveted connections.

Since a variance in the clamping force can occur from rivet to rivet in an actual structure, test results for reduced clamping should be considered as the lower bound for design. Category D of the AASHTO specification is intended

to provide a lower confidence limit to the available test data on riveted joints (11), as shown in Figure 2.1.

A study by Baron and Larson investigated the effect of clamping force and grip on the fatigue strength of riveted and bolted joints (6). The clamping properties were determined by removal of the plate material from between the rivet heads. For hot driven rivets the average clamping stress increased with length of grip. As the grips become longer the deformation of the rivet head, caused by shrinkage after driving, is a smaller part of the total shrinkage. However, the use of a long grip does not ensure that a high clamping force will be developed in the joint.

In 1963 Parola, Chesson, and Munse conducted a study into the effects of the bearing ratio on the fatigue strength of riveted connections (7). It was found that fatigue strength increased as bearing ratio decreased. (The bearing ratio is the relationship of the bearing stress on the rivet shank to the average net tensile stress. The bearing stress on the rivet shank is based on the thickness of plate times the nominal rivet diameter.) The reason the fatigue strength increased as bearing ratio decreased was that the stress concentration resulting from the load delivered to the plate by a rivet or bolt in bearing may be as much as twice that caused by the hole alone. This study mentions the conclusions reached in two other investigations. One showed that the fatigue strength of plates with open holes was greater than that for riveted

joints. The other found that plates with rivets had a fatigue life between that of plates with holes and plates with no holes. The conflicting results could be due to the clamping forces. The range of fatigue strength at a given life and bearing ratio is affected by the clamping force. Another consideration is that in order to alter the bearing ratio the transverse spacing, the plate thickness, and the rivet grip must be changed. It is hard to identify the proportional effects of these three factors.

Other possible factors may contribute to variations in the fatigue lives of riveted connections. Hole filling is one (7). When a rivet fills the hole well the load is distributed uniformly. However, when it does not, greater stress concentrations are produced, thereby reducing the fatigue life. Another factor is the drifting of holes which can occur during erection (12). When the holes in the member to be joined do not coincide, elongation of one side of the hole is produced by means of a drift pin. This is a likely site for initial cracks to develop and results in a reduced fatigue life for the joint. Eccentric bearing of a rivet has a similar effect and was found to be responsible for numerous failures in single-lap joints in the floorbeam hangers of railway bridges (13). Painting of the contact surfaces with red lead paint is also a factor that reduces the fatigue life (14). The paint has a low frictional resistance bringing the rivets in more bearing as compared to a milled surface. Despite all these other factors, the

clamping force appears to be the dominant factor.

2.2. Replacement of Rivets by High-Strength Bolts

The possibility of using high-strength bolts in steel construction was suggested by Batho and Bateman in 1934 (15). A study to compare the fatigue strength of bolted and riveted connections by Wilson and Thomas concluded that the fatigue strength of high-strength bolts was as great as that of well driven rivets (4).

The formation of the Research Council on Riveted and Bolted Structural Joints in 1947 (now the Research Council on Structural Connections), and the involvement of the American Railway Engineering Association stimulated the development of the high-strength bolt. In 1951, the Research Council issued its first specification. This specification permitted the rivet to be replaced by a high-strength bolt on a one-to-one basis. In 1960, after the test results of many studies had been presented to support the hypothesis that the fatigue strength of high-strength bolted joints exceeds the fatigue strength of riveted joints, the high-strength bolt was officially recognized by the Research Council on Riveted and Bolted Structural Joints as having strength properties superior to those of rivets (5,10,15,13,16). This superior strength is attributed mainly to the greater clamping force of a high-strength bolt.

Early demonstrations of the usefulness of high-strength bolts were in railway bridges. Rivets that had become loose

were replaced with high-strength bolts; after several years the bolts were still tight because of the clamping force.

In 1975 a study of the effectiveness of structural rehabilitation obtained by replacing rivets in critical locations with high-strength bolts was implemented by the Bethlehem Steel Corporation (8). Sixteen full-scale prototypes of a riveted truss chord to panel point connection typical of ore bridges, and two actual specimens from service, identical to the prototypes, were tested. The test results are included in Figure 2.1.

The tests were generally concerned with investigating the fatigue life of a riveted joint and a rehabilitated joint to determine the effectiveness and adequacy of joint rehabilitation by the technique of replacing the rivets in critical locations with high-strength bolts. Of the eighteen specimens only two of the prototypes were not subjected to constant amplitude loading. A variable amplitude loading was applied to these two specimens to determine if the fastener rehabilitation technique was as effective under a service simulation loading.

The tests showed that the fatigue lives of the joint specimens, which were rehabilitated when short cracks appeared, were two to six times greater than the longest lives observed for unrehabilitated joints. The critical fatigue locations were the same in both the prototypes and the actual specimens. When rivets were replaced only in the critical locations, the less highly stressed sites became

critical. It was found that the prevention of crack initiation at the less highly stressed sites as well as at the critical areas could be accomplished by replacing the rivets with high-strength bolts at both these locations.

It was concluded that this method of replacement of rivets with high-strength bolts at locations of observed or anticipated cracking is a way of significantly retarding subsequent crack growth and extending the fatigue life. The effectiveness of this technique of rehabilitation was maintained in prototype and actual specimens whether they were subjected to constant or variable amplitude loading.

2.3. Present Code Requirements

The AASHTO (11) and AREA (17) requirements for fatigue strength of riveted joints were based on the type of data shown in Figure 2.1. Most, but not all of the data were available at that time.

The fatigue provisions of the AASHTO specification classify riveted connections under category D, mechanically fastened connections with low slip resistance. If the rivets can be verified to be tight and to have developed a normal level of clamping force, then the use of a more liberal fatigue design case, Category C, is allowed. However, no further advice is given in the specification as to how a "normal" level of clamping force is to be established. The American Railway Engineering Asscіation (17), and American Institute of Steel Construction (18) have adopted the same

classification of this detail as AASHTO.

Bolted connections are placed in Category B by both the CSA and AASHTO standards.

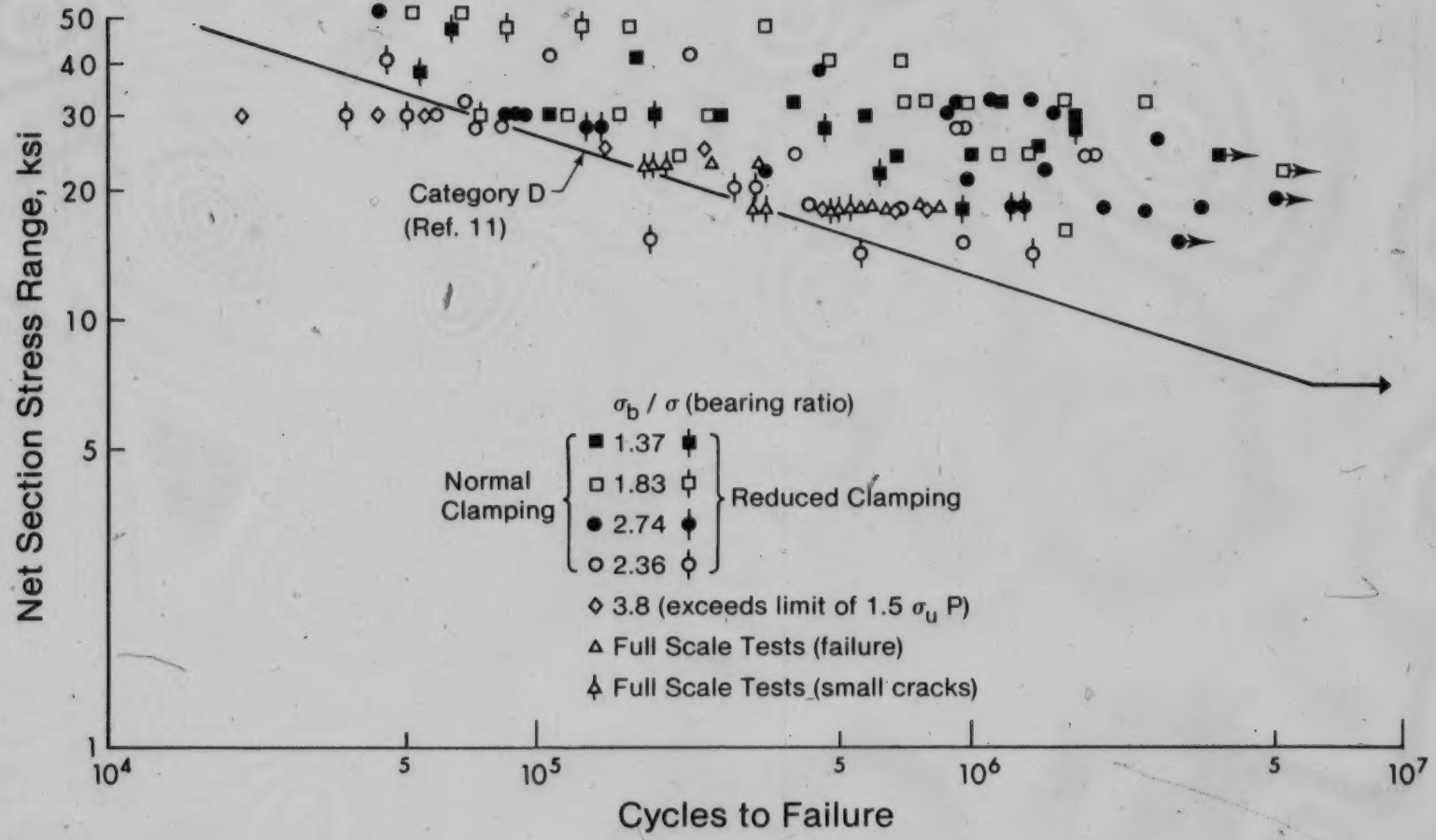


Figure 2.1 Summary of Test Data on Riveted Joints

3. EXPERIMENTAL PROGRAM

3.1. Scope

In this project ten tests were performed: three of these were used to examine the fatigue strength of joints with completely loose rivets, three to study the effect of using high-strength bolts to replace rivets in critical locations of joints, and the remaining four to examine the fatigue strength of actual riveted connections taken from existing structures.

The joints with loose rivet connections were simulated by simply punching holes in a plain beam. The holes were punched rather than drilled as fatigue can be expected to be more critical in punched holes. This condition was intended to represent the lower bound of riveted joint behavior, that is, the rivet is completely loose. Obviously this model neglects the beneficial effects of any clamping force the rivet could provide. On the other hand the detrimental effects of fretting or corrosion under a rivet head that might be present in the prototype are also neglected.

Plain beams with high-strength bolts placed in punched holes were used to study the effect of renovating riveted joints by the replacement of rivets with bolts.

The fatigue strength of riveted connections was examined by testing four riveted members taken from a seventy-year-old bridge.

3.2. Specimen Description

3.2.1. Test Beams

The six test beams were obtained as surplus from a previous test program. This was advantageous as the basic fatigue capacity of these beams had already been established.

The specimens were all W200 x 36 shapes of CSA G40.21-M 300W steel (19). The test beams were all 3.2 m long. Each specimen had eight 21 mm diameter holes punched in the tension flange (four holes on each side of the web). They were located symmetrically at midspan with a center-to-center spacing along the beam of 60 mm (Fig. 3.1).

Three of the specimens had ASTM A325 3/4 in. (approximately 19 mm) diameter bolts 3 in.(76 mm) long installed in the holes. Figure 3.2 shows the beam with empty holes and with bolts in the holes. The turn-of-nut tightening method was used to install the bolts (20).

3.2.2. Riveted Members Taken from Service

The riveted specimens were obtained from a section which was being replaced in a seventy-year-old highway bridge (Fig. 3.3). The condition of the bridge members was excellent. There was no evidence of flaking paint or corrosion of the surfaces. Each member consisted of either two 3 in. x 5 in. x 0.375 in. angles (76 mm x 127 mm x 10 mm) or two 3.5 in. x 6 in. x 0.375 in. angles (89 mm x 152 mm x 10 mm). The short legs were placed back-to-back, with a

space between the legs of 8 mm (Fig. 3.4-Type A and 3.5-Type B). Each specimen was between 3.0 m and 3.5 m long and had pairs of rivets located as is shown in Figures 3.4 and 3.5. Since the material properties were not known, coupon tests were conducted. Four coupons were made from short, loose pieces of the material. The average static yield strength of these specimens was found to be 241 MPa.

In order to test the bridge specimens, the outstanding legs of the angles were reduced to 1.5 in.(38 mm) and a connection was designed to facilitate the use of clevis and pin attachments that could be used in the testing machine.

3.3. Test Set-Up

3.3.1. Test Beams

All six specimens were loaded in the same way. The beams were tested on a 3.05 m span with the test section, that is the holes or holes with bolts, contained within a region of constant moment. A spreader beam 0.91 m long distributed the applied load symmetrically at two points to establish the constant moment region (Fig. 3.1).

Steel rockers were placed at the two loading points and at the reaction points of the test beams to establish simple supports. The spreader beam was laterally braced during the testing.

To provide the desired repetitive load to the test piece, a Pegasus servo-load simulation system was utilized.

A servovalve, manipulated by a controller, regulates the application of hydraulic power to an actuator, thus producing the required force. The jack has a maximum dynamic capacity of 489 kN. In conducting the tests loading speeds between 210 and 300 cycles/minute were used.

Four strain gauges were mounted on the underside of the bottom flanges. Figure 3.1 shows the location of the gauges. The strains were established after the stress range and the minimum stress had been selected. From the strains, corresponding to the minimum and maximum stresses, the minimum and maximum loads were established . The loads were then applied and monitored on the Pegasus controller.

3.3.2. Riveted Members Taken from Service

The bridge members were placed vertically in the test set-up. In order to accomodate the specimen in the testing machine an 8 mm x 380 mm plate x 760 mm was partially sandwiched between the two angles at each end. It was held in place by four 7/8 in.(22 mm) diameter bolts 3 in.(76 mm) long. The line of bolts was offset 13 mm from the centerline of the plate to allow for the eccentricity of the specimen. The parts of the plates which extended beyond the ends of the specimen were reinforced by two plates. An 89 mm diameter hole was drilled at the centerline of the plates for insertion of the pin (Fig. 3.6). Figure 3.7 shows a specimen installed in the testing machine.

A MTS universal testing machine, with a maximum tensile capacity of 6675 kN, applied the cyclic axial tension load to the specimen at loading speeds between 36 and 60 cycles/minute.

Six strain gauges were positioned on the specimen, three on each angle in line with the rivets near the top, at the center and near the bottom. The same procedure that was used for the six test beams was followed to establish the minimum and maximum loads for these tests.

3.4. Testing Procedure

3.4.1. Test Beams

All six beams, three with bolts and three without bolts, were tested at one stress range level. This was done as it was considered necessary to acquire data only on the shift of the stress range versus number of cycles line as compared to the standard case. In other words, the slopes of the fatigue response curves of the plain beam, the beam with holes, and the beam with holes containing bolts are assumed to be equal. The stress range chosen was 216 MPa. This was selected on the basis of testing time, yield point of the specimen and the number of specimens available. Considering all of these factors, it was decided to develop data for one region, the higher end of the stress range versus number of cycles design curve. The minimum stress was 28 MPa and the maximum stress was 244 MPa.

Failure of the specimen was determined by an increase in midspan deflection. From previous fatigue studies the crack size considered to cause failure of a section corresponded to a deflection of 0.5 mm (21,22). When this deflection limit was achieved the Pegasus system automatically stopped.

Testing proceeded intermittently for periods ranging from several hours to a few days in order to allow examination of fatigue crack propagation. After the fatigue testing was completed the damaged areas were flame cut from the beam. The fatigue cracks were then saw-cut open and the failure surfaces examined.

3.4.2. Riveted Members Taken from Service

The testing of the bridge members was done at one stress range, 138 MPa. The stress range selected was based on the maximum stress that would be attainable without yielding the specimen and the minimum stress range that was feasible considering the expected testing time. The minimum and maximum stresses were 20 MPa and 158 MPa, respectively.

It would have been desirable to test these specimens at the same stress range as the six test beams but because of the yield point of the specimen this was not possible. However, with results from other documented tests (8,17) and this entire investigation, the six beam and four bridge member tests, a comparison could be made of the shift of the stress range versus number of cycles line to the predicted

line.

When the specimen had fractured, the MTS system ceased to operate. After failure had occurred at one of the rivets the remaining specimen was prepared at the ends for further testing. The testing was then resumed until a second failure occurred. This procedure was continued until the specimen could no longer be tested.

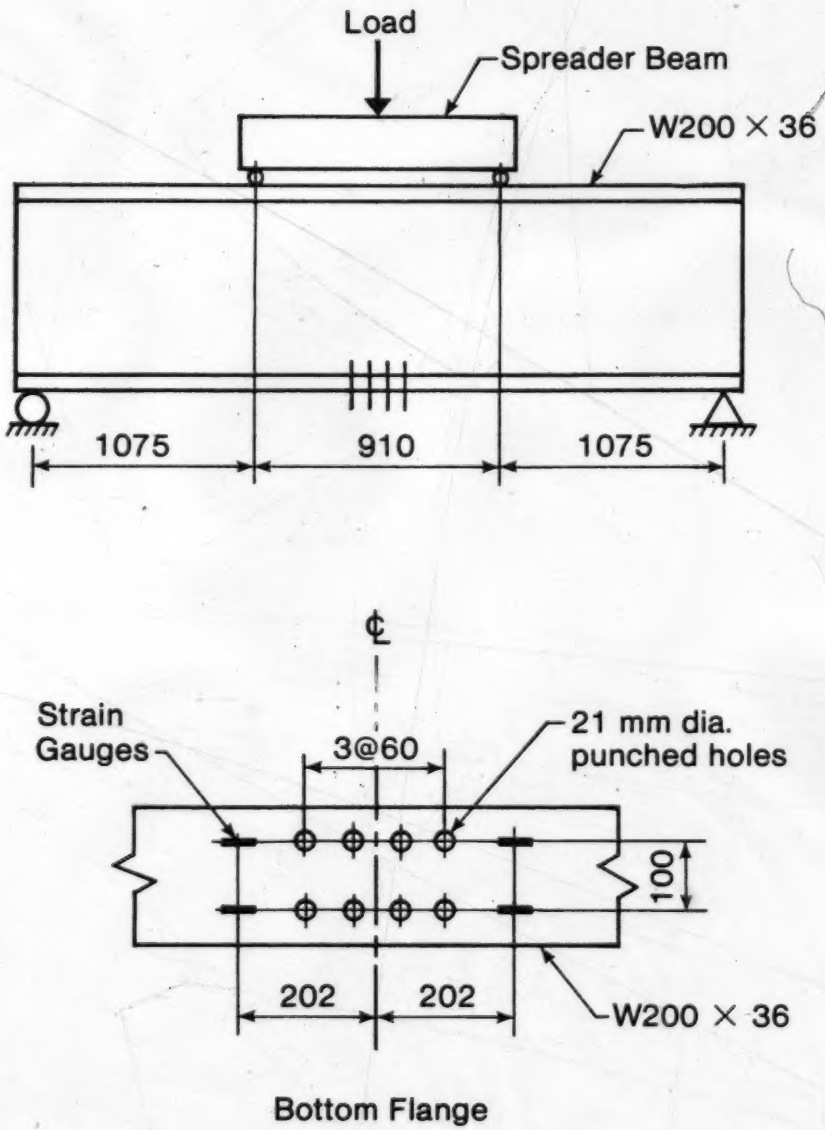


Figure 3.1 Specimen Details for Test Beams

Figure 3.2 Typical Test Specimens of Beams with Holes and Beams with Bolts

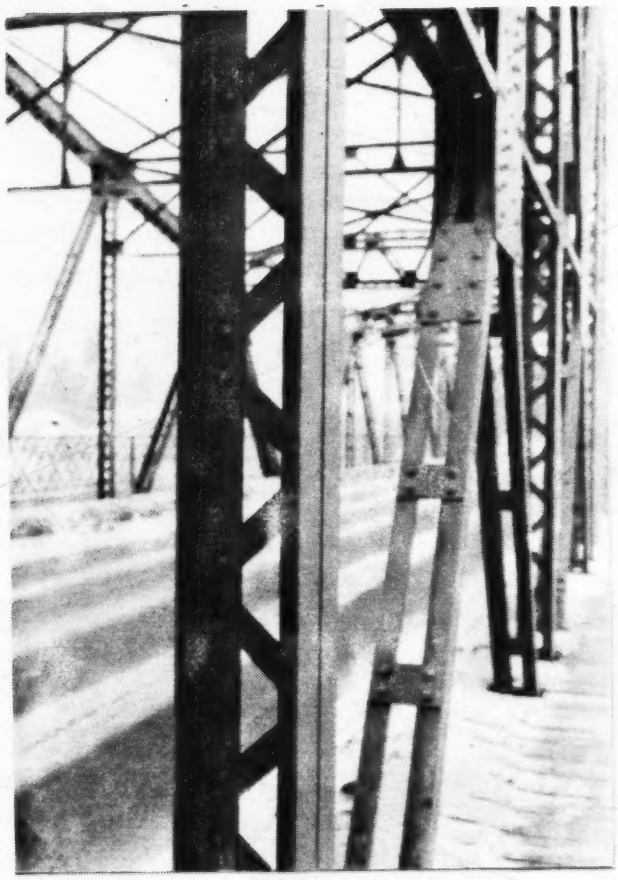


Figure 3.3 Typical Bridge Member

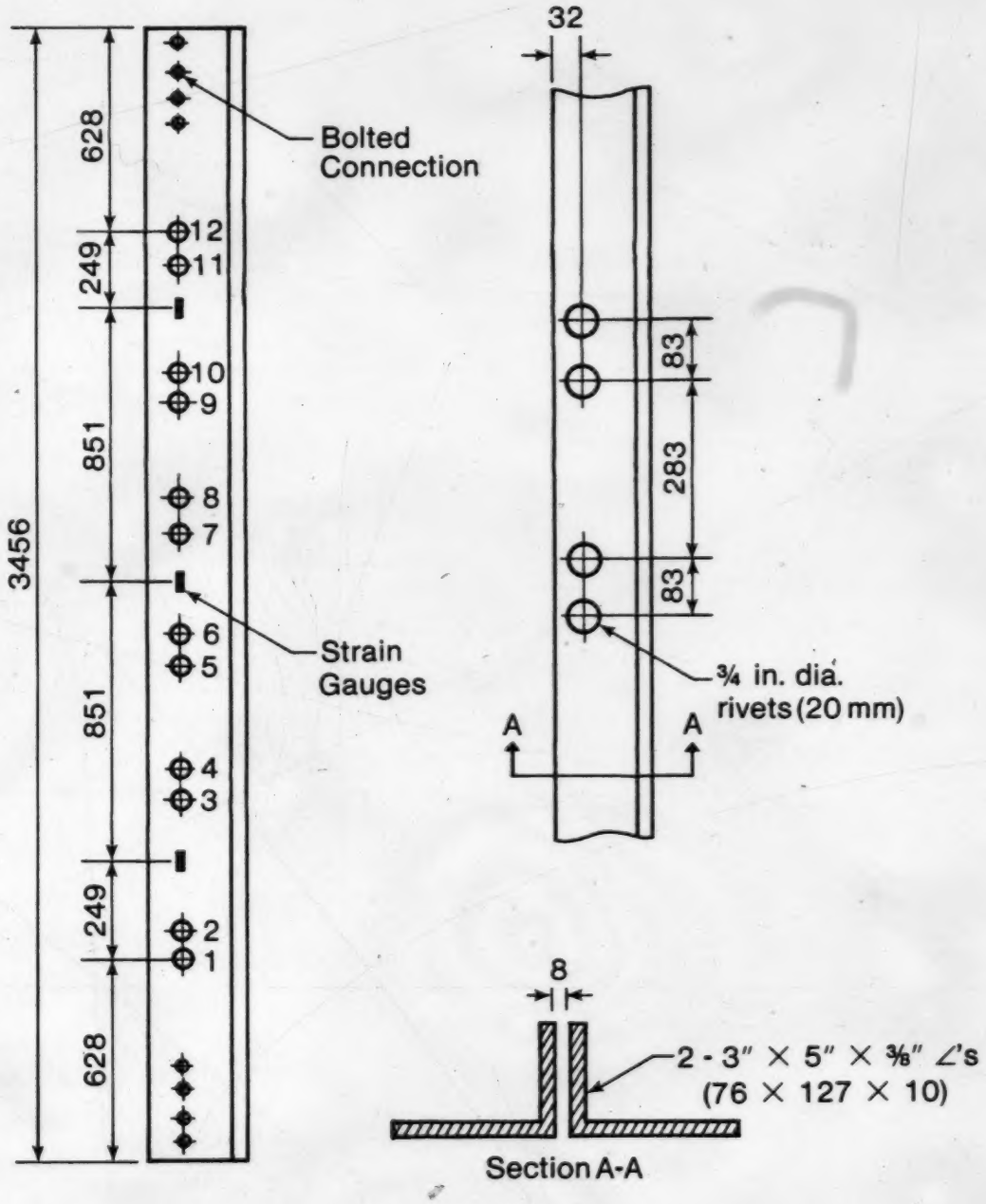


Figure 3.4 Specimen Details of Bridge Members Taken from Service

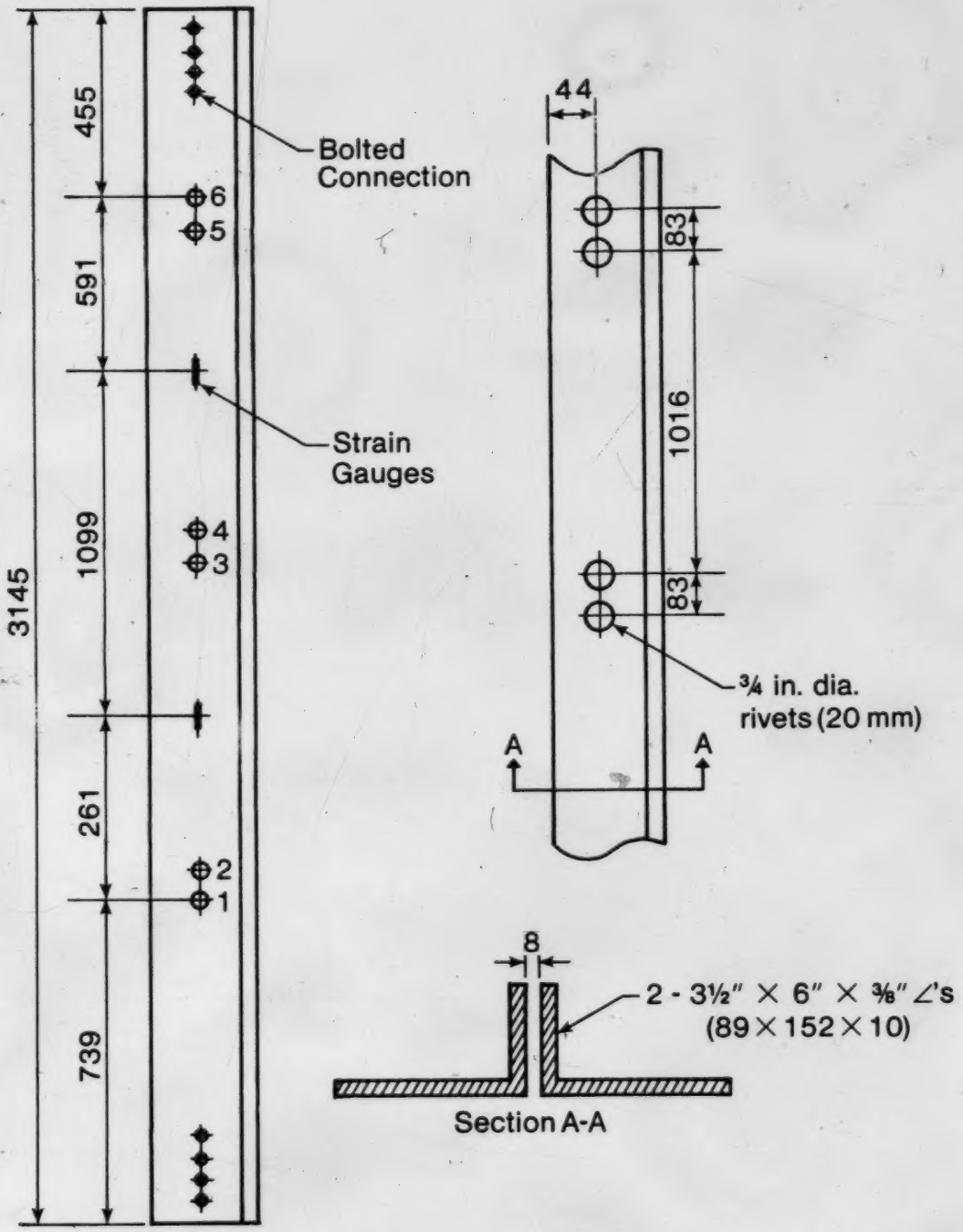


Figure 3.5 Specimen Details of Bridge Members Taken from Service

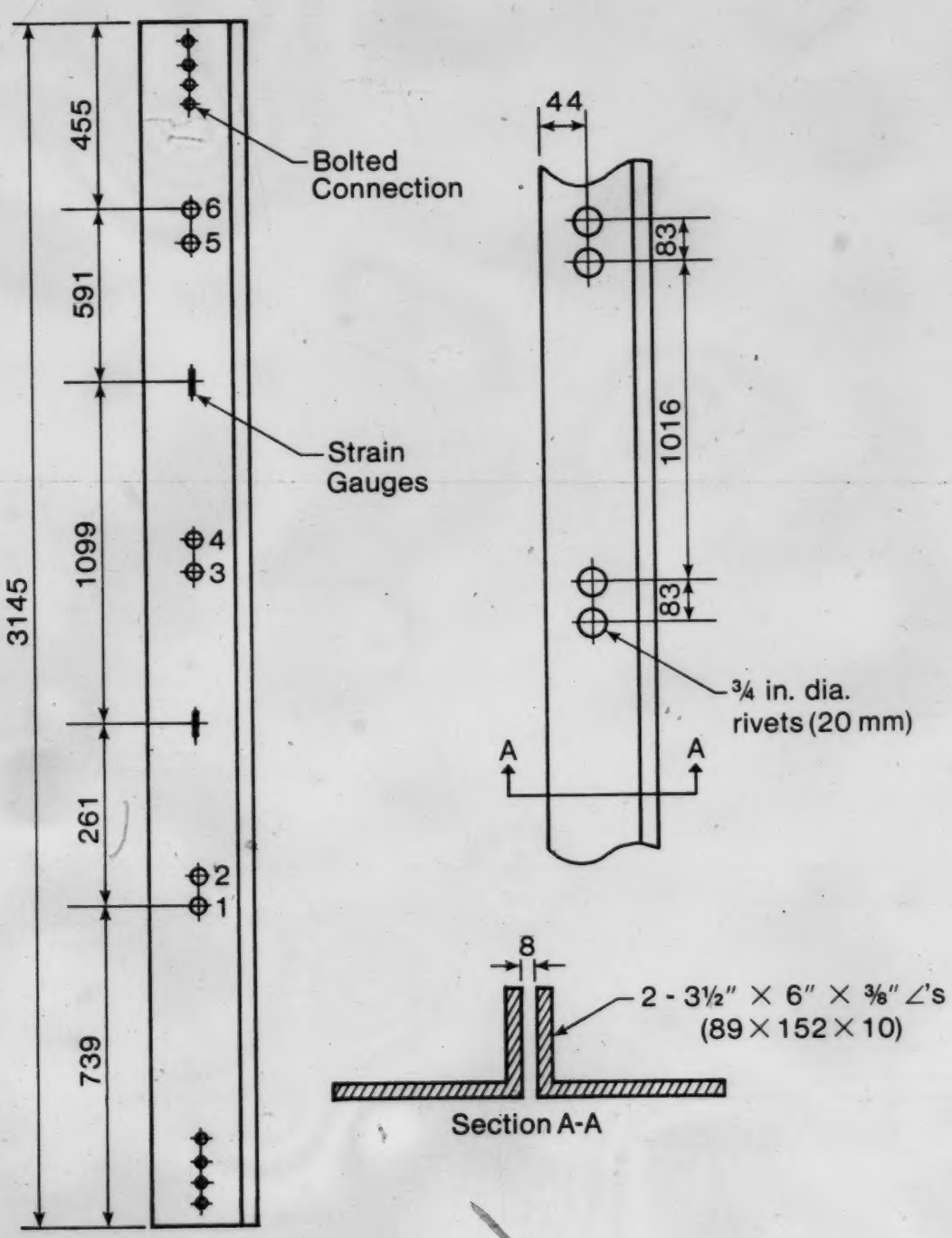


Figure 3.5 Specimen Details of Bridge Members Taken from Service

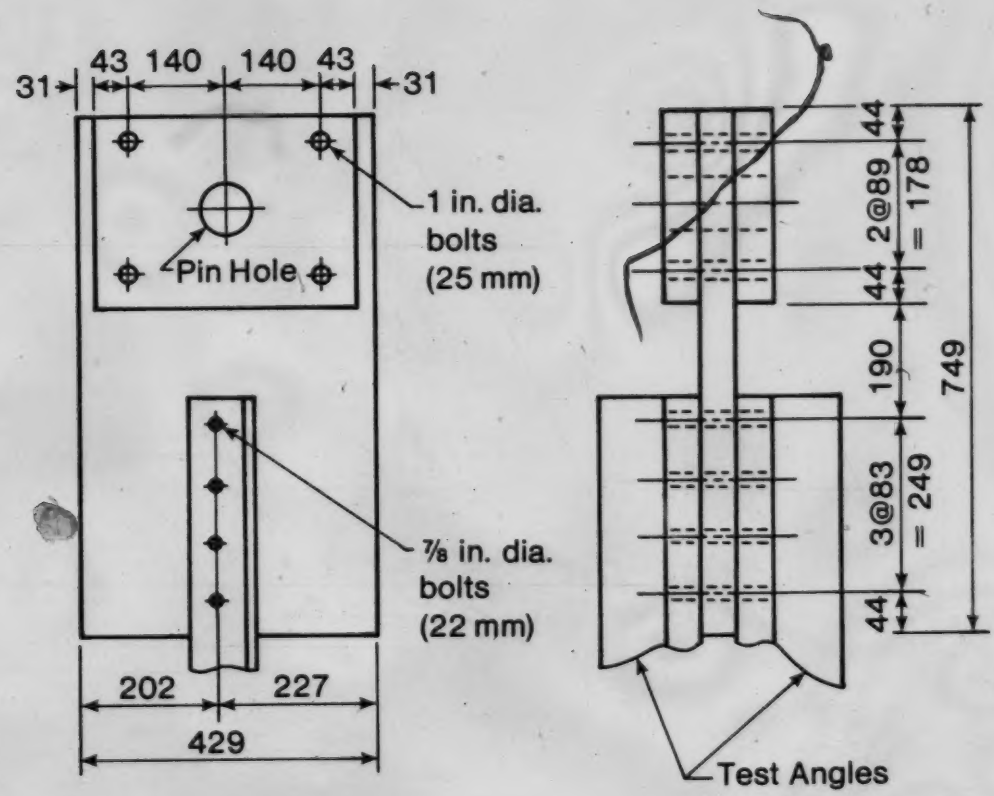


Figure 3.6 Connection Details for Bridge Specimens

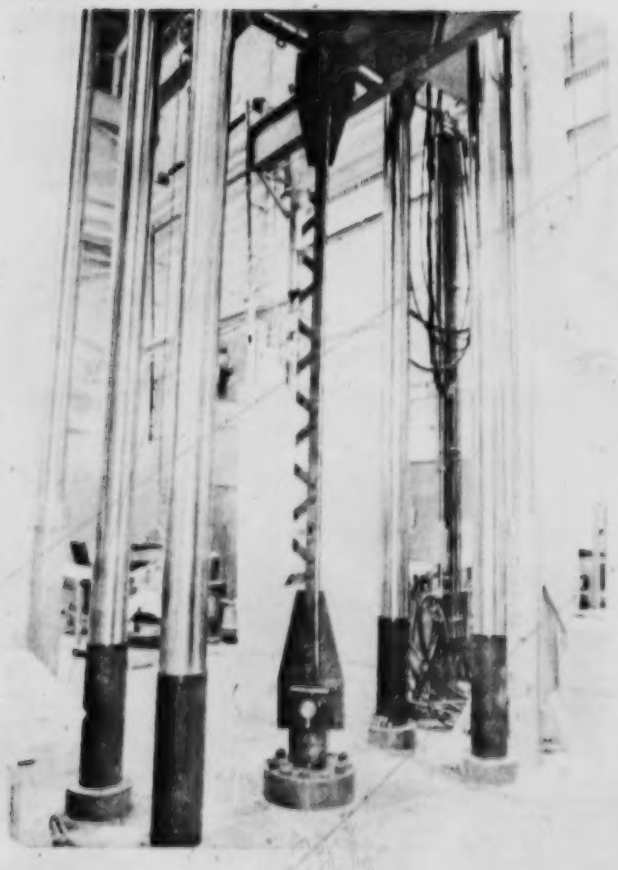


Figure 3.7 Bridge Specimen Installed in Testing Machine

4. TEST RESULTS

4.1. Crack Initiation and Growth

4.1.1. Test Beams

In five of the six tested specimens, three with open holes and two with holes containing bolts, the crack initiation and growth patterns were similar. Initial cracking began at the extremity of one of the end holes and proceeded to grow out towards the flange tip (Fig. 4.1). Before the crack had progressed from the hole completely through to the flange tip, another crack started on the opposite side of the same hole and progressed in towards the web. In the sixth specimen, the crack started at the extremity of an interior hole and grew out towards the flange tip.

Only one of the beams, one with open holes, had cracks occurring at other holes in addition to those at the hole that led to failure. In this case, the cracks that appeared at other holes were all located on the same side of the web as the hole that eventually caused failure.

Failure was considered to have occurred when the dynamic deflection increased by 0.5 mm. The cracks generally had caused a reduction of 25 to 50% of the flange cross sectional area by this time.

Several of the fracture surfaces were viewed under an electron microscope. (Fig. 4.2). Observations in the

Bethlehem steel tests on riveted and rehabilitated full-scale model joints showed differences in the rate of crack growth, the farther apart the fatigue markings, the greater the rate of crack growth (8). No such comparison can be made between the rate of crack growth in the beams with open holes and in the beams with high-strength bolts filling the holes due to the uncertainty of the specimen orientation in the electron microscope chamber when the photographs were taken.

4.1.2. Riveted Members Taken from Service

Since each bridge member had several riveted details within its length, more than one result was obtained from each specimen. This was accomplished by successively removing a failed section and establishing a new end connection. Table 4.2 presents the test results and the failure locations. The failure location given in the table provides the type of failure, and rivet hole at which the crack occurred. The four test pieces, therefore, produced a total of eleven fatigue strength results.

In all of the specimens but one, the fatigue crack that developed grew normally to the applied stress at the extremity of the rivet hole and progressed out to the edge of the angle leg that contained the rivet. Before this crack had progressed completely to the edge another crack started from the opposite side of the same hole and moved in towards the corner of the angle. Figure 4.3 shows a typical fatigue

crack in a rivet hole. The exception was a failure at a flaw on the edge of an angle leg where the surface had been ground to reduce the size of the specimen. This was not included in the results. It should be noted that three other failures at rivet holes occurred in the same specimen before a failure crack was initiated at this flaw. A fractographic and microstructural analyses were performed on one of the specimens that had a fatigue life shorter than some of the others. A section was made perpendicular to the plane of the crack in the rolling plane. It was then ground, polished, etched with nital and examined. Nothing out of the ordinary was found. The steel had a carbon content of 0.2%, many non-metallic inclusions, and was considered dirty by modern standards.

At failure, the cross section had been reduced by 30% to 100%. Since the fatigue crack begins at the hole it cannot be seen until it grows beyond the rivet head. Cross sections which were reduced by 100% were the result of the tests being run 24 hours a day. However, in most cases the test was stopped when one angle of the pair fractured. It should be noted that riveted built-up members provide redundancy; a crack that forms in one component cannot directly grow into one of the other components in the structure.

4.2. Factors Affecting Failures

4.2.1. Test Beams

A pattern was observed in the location of the cracks that eventually caused failure. In every case but one the failure fracture occurred at an end hole, even though all the holes were within the constant moment region. The observed behavior is consistent with previously recorded failures of riveted connections (8,13,23,12). This occurs because the stress concentration at the end hole is greater.

The failure location was always on the side of the flange where the distance from the hole to the flange edge was the least. On a given side the edge distance varied by only 0.25 to 1.27 mm. However, from side to side the variance in some cases was as much as 5 mm. This diversity can be attributed to the holes being punched rather than drilled.

The test data, summarized in Table 4.1, confirms that beams in which the holes are filled with high-strength bolts are superior in fatigue to beams with open holes, the prototype for loosely riveted holes. The fatigue life of the specimens using high-strength bolts is from 14 to 20 times greater than the specimens with open holes for the same stress range. This can be attributed to the clamping force provided by the bolt.

4.2.2. Riveted Members Taken from Service

The test results obtained from the testing of the riveted members had a noticeable scatter, just as was found with the collected data on riveted joints (Fig. 4.4 and Fig. 2.1). One of the factors responsible for the variance in the fatigue lives is probably the effect of the clamping force. An attempt was made to determine the clamping forces in the rivets but the degree of precision was too small to be useful.

The location of the cracks which caused failure had no pattern. They occurred in riveted details located in various positions of the specimen.

The clamping force has been shown to be affected by the length of grip (6). In this investigation all the rivets were the same size, 3/4 in. in diameter (20 mm) and had a grip length of 1-3/16 in. (30 mm). Figure 4.5 shows that there is a great variance in the clamping stress for this length of grip. This, with the conclusions reached in other studies (5,6,7,12), substantiate the importance the clamping force has on the fatigue life of riveted connections. This could explain why several of the fatigue lives were so great.

Another factor that could have an effect on the fatigue life is the manner in which the specimens were tested. Since the tensile loads were applied to the angles rather than to the bridging between the angles, the rivets could not slip and bear against the edge of the hole and cause a local

stress. Some field connections will behave this way, either because the rivet does not come up against the edge of the hole, or because the rivet does not transmit force. In a member containing lacing bars, such as the ones from which these test pieces were taken, the force transmitted by the lacing bar to the rivet is normally very small. Nevertheless, the main member contains a riveted detail. The fatigue life of members containing rivets which are actually pulled up against the connected, loaded parts might be expected to be less than those recorded here because of the effect of the stress concentration.

The previous stress history of the riveted specimens was analysed to determine if it had an effect on the fatigue life. The bridge geometry and member measurements were obtained from field measurements (Fig 4.6). From this information the influence lines were drawn for the members being considered, those labelled as A and B in Figure 4.6. In order to obtain the maximum stresses an HS 20-44(MS 18) truck was applied (11) and located at the critical location. The truck loads were increased by 30% to allow for impact and the truss nearest the lane with the truck was considered to take 75% of the load.

The maximum stress ranges for this loading were found to be between 41.4 MPa and 44.8 MPa depending on the bridge member. At these stress ranges the fatigue life of a riveted detail (17) is theoretically unlimited. Therefore, the previous stress history was assumed to have a negligible

effect on the fatigue life.

Despite this conclusion a duty spectrum was tabulated and Miner's Rule applied (24). It was assumed that the bridge was used on a daily basis by ten trucks from 1912-1951, by thirty-six trucks from 1951-1966 and by seventy-two trucks from 1966 to the present. This is based on known bus route histories, type and size of bridge and type of bridge traffic. This increased the fatigue life obtained from the riveted specimen tests by 5%. This is insignificant as it does not suggest that a riveted detail subjected to fatigue should be designed by any other category other than Category D (Fig 4.4) of current specifications (17,20,11).

4.3. Comparison with Previous Studies

The Bethlehem Steel Corporation conducted a study on the effectiveness of rehabilitation of riveted connections by the replacement of rivets with bolts (8). The outcome of the unrehabilitated tests on full-scale models of riveted joints and an actual riveted joint are plotted in Figure 4.4 for comparison with the findings on riveted specimens. They all fell to the right of Category D, just as the results obtained herein did, indicating that the rivets had provided some clamping force.

Thus, from the test data presented, when a clamping force is supplied by the rivet, Category D will give a satisfactory prediction of the fatigue life but for members

where rivets are loose, modelled by beams with holes, Category D slightly over-estimates the fatigue life. In all cases, it is assumed that the condition of the members is such that corrosion will not affect the results. It should be emphasized that the bridge members taken from service and tested in this program were in excellent condition.

The results of the tests on beams with high-strength bolts filling the holes were compared with Category B, the classification for bolted connections of the CSA standard (20). They were found to fall well to the right of the line for Category B. In fact, the test data were found to approach Category A, the condition for plain material. (The category lines represent the 95% confidence limit for 95% survival.) These test results for beams with bolts endorse Category B as a safe way of predicting the fatigue life of bolted connections.

The fatigue lives of the beams with bolts were also collated with the tests performed on rehabilitated full-scale models of riveted joints by the Bethlehem Steel Corporation (8). In the Bethlehem tests, the rehabilitated riveted joints, joints where rivets were replaced with high-strength bolts at the first sign of a crack, had an increased life over the full-scale models of riveted connections. In some cases these rehabilitated joints had an improved life comparable to that of the beams in this investigation which had high-strength bolts filling the holes.

In examining the tests on beams with open holes, the prototype for rivets with no clamping force, the fatigue lives were less than those of the riveted details. The data points fall slightly to the left of the AASHTO Category D (17). Category D is the class for members with riveted connections of low slip resistance. This might be reasonable as there is no clamping force in the specimen tested; actual riveted members might have at least a small clamping force.

The data presented here confirms that substantial benefits are gained from replacing rivets with high-strength bolts in critical locations of connections.

Table 4.1 Summary of Results for Test Beams

Specimen	Stress Range (MPa)	Observed Crack (cycles x10^3)	Failure (cycles x10^3)	Crack Initiation
BH-1	216.1	56.4	58.6	E
BH-2	216.1	———	45.9	E
BH-3	216.1	45.5	50.0	E
BB-1	216.1	887.1	907.3	E
BB-2	216.1	840.0	845.6	E
BB-3	216.1	———	687.0	M

In all cases, σ_{min} =27.6 MPa

BH=beams with empty holes
BB=beams with high-strength bolts filling the holes from service
E=extremity of end hole
M=extremity of middle hole

Table 4.2 Summary of Results for Riveted Bridge Members

Specimen	Stress Range (MPa)	Failure (cycles x10^3)	Crack Initiation and Location
RS-1a	137.9	418.5	R A8
RS-1b	137.9	418.5	R A10
RS-1c	137.9	958.3	R A6
RS-2a	137.9	782.3	R A10
RS-2b	137.9	793.6	R A2
RS-2c	137.9	911.8	R A5
RS-3a	137.9	263.1	R B2
RS-3b	137.9	496.3	R B3
RS-4a	137.9	249.1	R B6
RS-4b	137.9	352.8	R B2
RS-4c	137.9	485.2	R B4

In all cases, σ_{min} =20 MPa

RS=riveted bridge members taken from service
R=rivet hole
F=flaw in ground edge of angles
A=type A specimen Number indicates
B=type B specimen location. See Figs. 3.4 & 3.5

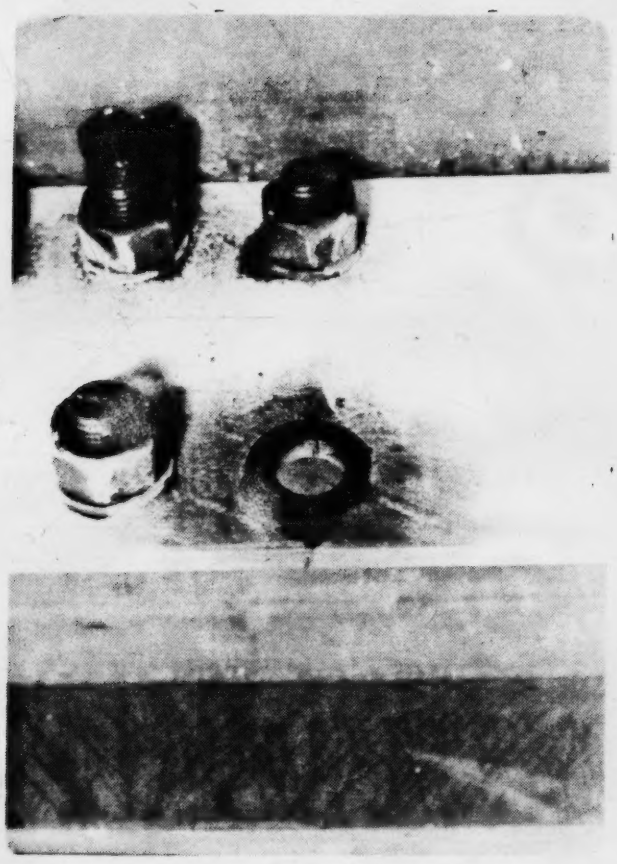


Figure 4.1 Failure Crack in Bolted Test Beam

Figure 4.2 Fatigue Striations(x3000) a)Beam with Open Holes
b)Beams with High-Strength Bolts filling the Holes

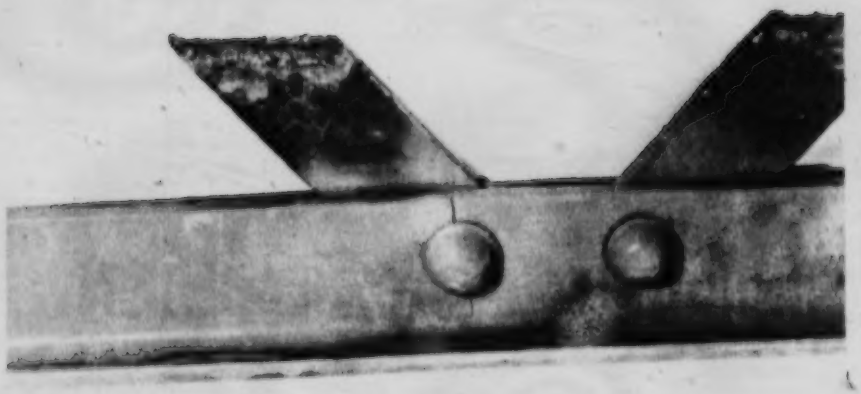


Figure 4.3 Failure Crack in Rivet Hole

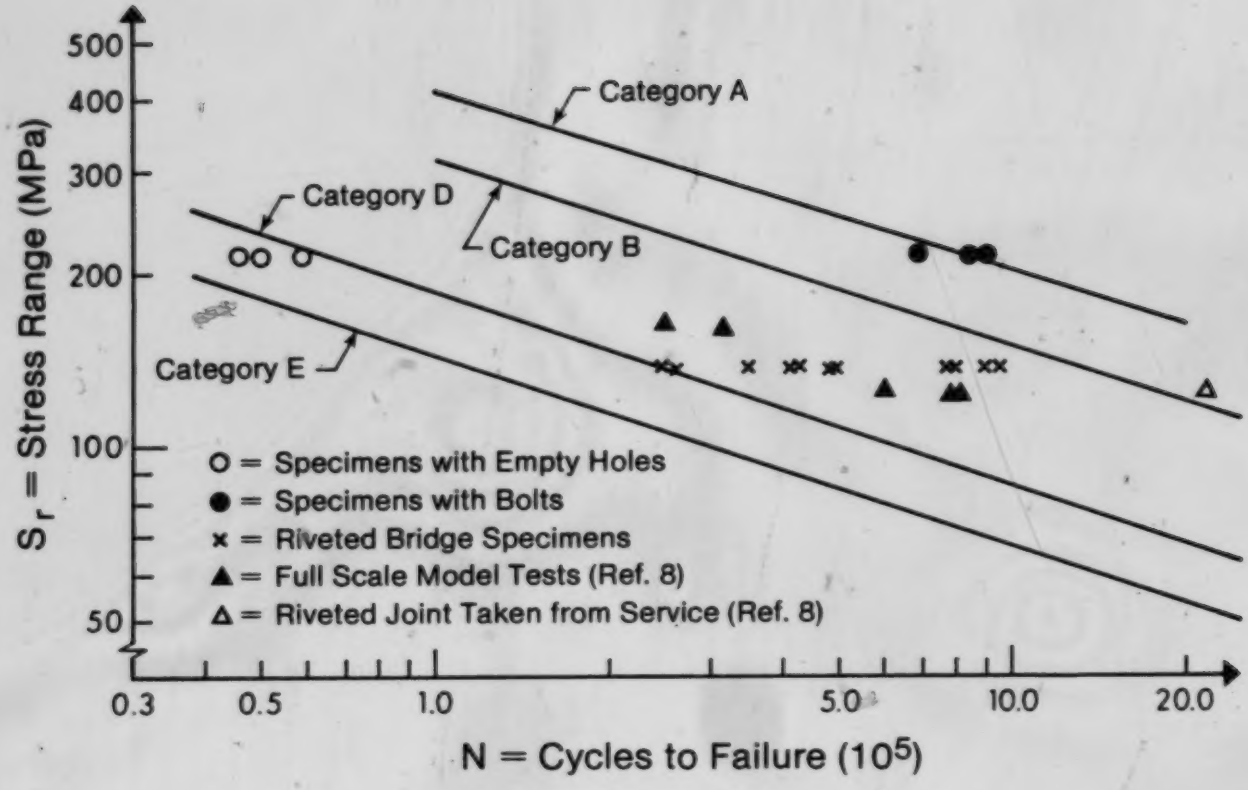


Figure 4.4 Test Beams and Riveted Members Taken from Service

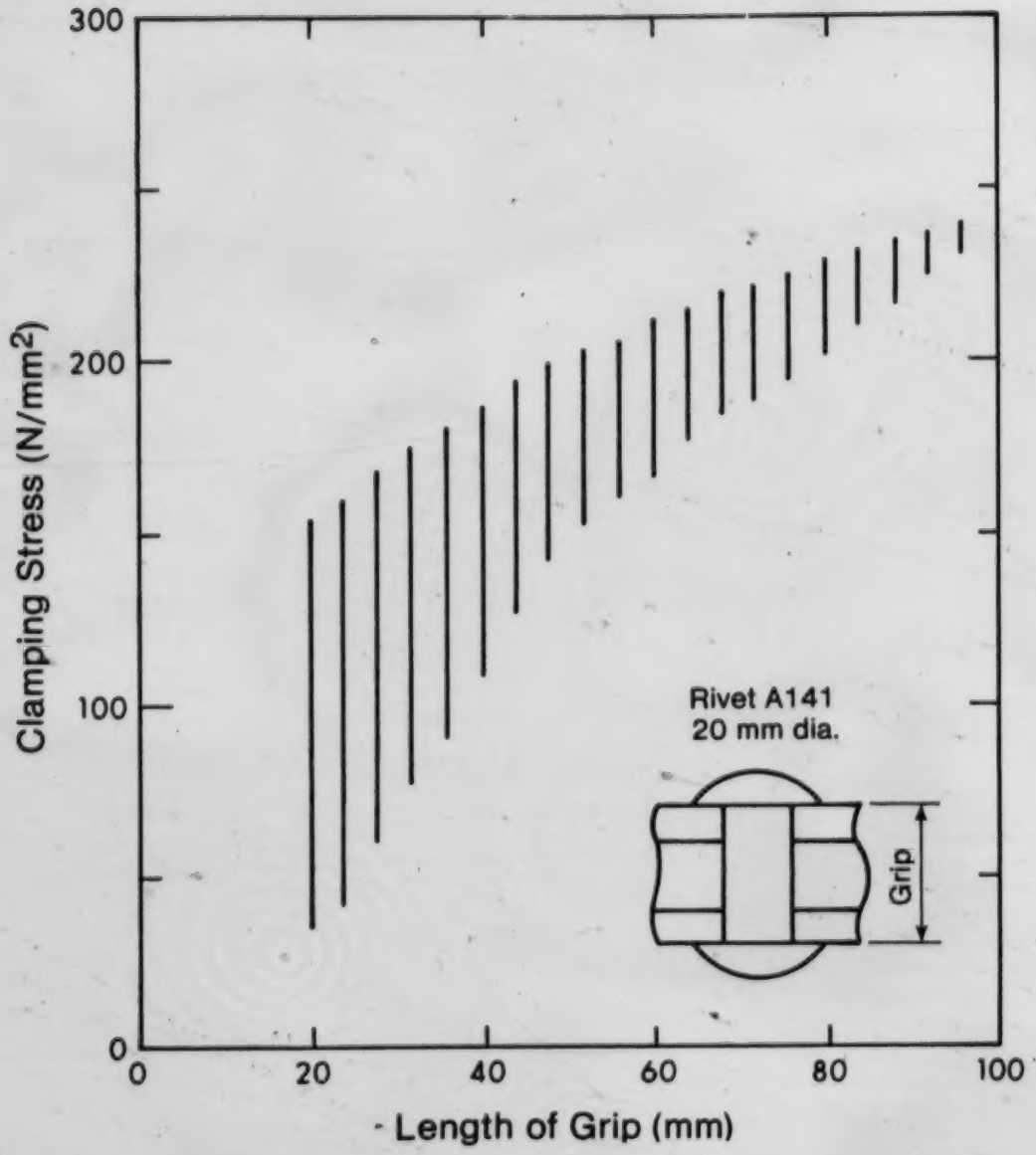


Figure 4.5 Clamping Stress versus Length of Grip (Ref. 6)

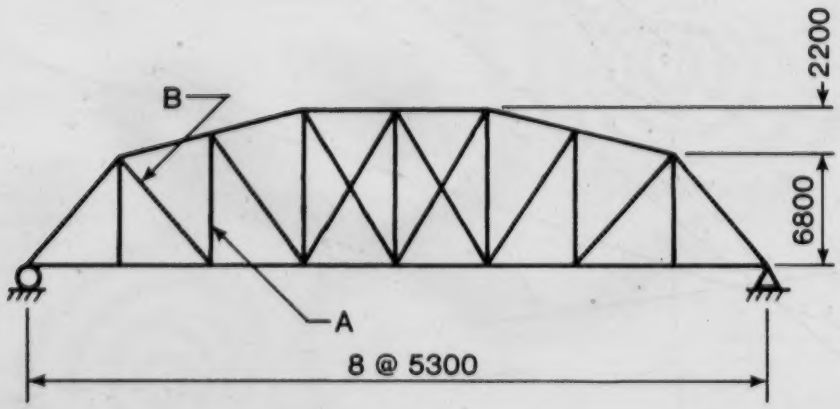


Figure 4.6 Bridge Geometry and Location of Actual Riveted Members Tested

PART B - GROOVE WELDS WITH BACKING BARS

5. LITERATURE SURVEY

5.1. Previous Studies

There is a paucity of information available on the fatigue behavior of groove welds with backing bars located transversely to the direction of applied stress. Although fatigue testing has been carried out on groove welds and transverse non-load carrying fillet welds (25), the effect of discontinuous fillet welds on the fatigue life of a groove weld with backing bar detail has not received much attention. This weld detail is prohibited for fatigue situations in some standards, and others categorize it more on the basis of theoretical findings and engineering judgement than on experimental results.

A theoretical study by Gurney (26) using finite element analysis shows that groove weld size, main plate thickness and attachment thickness influence the fatigue life of this type of detail. The extent to which these parameters affect fatigue life remains to be investigated.

5.2. Present Code Requirements

Current fatigue design requirements typically used within North America are presented in CSA (20) and AASHTO (11) standards. The rules presented are based primarily on a

series of tests conducted in the early 1970's for the National Cooperative Highway Research Program (21,22). From the results of these tests, theoretical analysis, and engineering judgement, structural details have been assigned to different fatigue categories depending on their fatigue life characteristics.

In the present CSA standards, Clause 12.5 of the CSA Standard W59-1977 (2) is the only clause that refers to the situation of a groove weld located transversely to the direction of stress. This clause requires the backing bar to be removed and the joint ground or finished smooth. Presumably, if this detail was permitted it would be designated as Category E, the class for intermittent fillet welds, as opposed to Category B, the class for a complete penetration groove weld ground flush.

For the detail of a groove weld with backing bar perpendicular to the direction of stress, the AISC specification (18) refers to the AWS D1.1-Rev 2-77 for details of welding procedures and other specifics (27). The AWS specification handles this detail in the same way as CSA W59 (2).

The British Standard BS 5400, Part 10 lists specific rules for groove welds with backing bars (28). When the weld and backing bar are perpendicular to the line of stress the detail is classified as Class E but if the backing strip is fillet or tack welded to the plate the detail is classified as Class F. The fatigue strength can be reclassified to

Class G if the backing bar is permanently tacked within 10 mm of the member edge. For 2 x 10^6 cycles, for example, the design stress is 68 MPa for Class F and 50 MPa for Class G. The background for these choices appears to be based on work by Gurney (26), as well as corrected experimental results of transverse groove welds made on a backing bar and transverse non-load carrying fillet welds (25). The results were modified to account for the fact that laboratory specimens were too small to hold full tensile residual stresses (25). Only information obtained since 1950 and tests conducted using uniaxial tensile loading or unidirectional loading have been considered in the classification of the joint (29).

An investigation by Thelen, Henschel, Neumann and Nieme dealt with the fatigue behavior of groove welds in which the permanent backings for the molten pool were attached in different ways (30). In this study the backing bars were either fixed on both edges, fixed on one edge or fixed at the root of the groove weld. They were all subjected to a pulsating tensile loading. Evaluation of the test results demonstrated no differences in the fatigue lives for the three different cases. Although the fatigue lives were higher than those specified in the East German code the difference was not substantial enough to move the detail into a higher classification.

The British, East German, West German and Italian codes all give fatigue requirements (31) for the detail of a

groove weld with a backing bar located transversely to the direction of stress where the backing bar is attached only at the root of the groove weld. It is therefore of interest to examine this detail. The rating indicates how good or bad the detail is in relation to its position in each standard. The agreement between the standards on the rating of this detail is compatible. However, the actual experimental results that the design stresses are based on differ considerably (31). This is due to different experimental foundations and safety factor concepts. Table 5.1 provides the comparative data for the different code classifications for this detail when subjected to 2×10^6 cycles of pulsating tension.

Table 5.1 Code Comparison for a Groove Weld with the Backing Bar Attached at the Root of the Groove Weld (Ref. 31)

Code	Allowable Fatigue Stress (N/mm^2)	Experimental Fatigue Stress (N/mm^2)	Rating
E. Germany -Code 13500	101	117	6/7
W. Germany -DV 952	113	169	4/6
Italy	99	–	5/7
Britain	100	100	6/8

6. EXPERIMENTAL PROGRAM

6.1. Scope

The purpose of the experimental study was to evaluate the fatigue strength of a groove weld with backing bar, the latter held in place by intermittent fillet welds. The entire detail is oriented transversely to the direction of the applied stress. The experimental program consisted of testing twelve specimens at various stress ranges.

6.2. Specimen Description

The groove weld with backing bar specimens were fabricated by a local steel fabricator. The fabricator was provided with material specifications and fabrication instructions.

Two 13 mm thick steel plates 700 mm x 3710 mm, from the same source plate, were groove welded along their length(the 3710 mm direction) using a continuous backing strip. The backing strip, 7 mm thick by 38 mm wide, was attached by 5 mm fillet welds 40 mm in length. The fillet welds were spaced at 150 mm and staggered on opposite sides of the backing bar. The requirements for the CSA W59 prequalified weld M2-2 (2), with a reinforcement angle of 45°, were followed in constructing the full penetration groove weld. The welds were made manually by the submerged arc welding process using AWS E7018 electrodes. The groove weld was inspected using the X-ray method.

After the groove weld was made, the main plate was then cut into 300 mm wide strips to make twelve specimens each 13 mm x 1400 mm x 300 mm (Fig. 6.1). The specimens were identified according to their location in the original plate.

In order to accomodate the specimens in the test set-up, ten 24 mm diameter holes were drilled at either end of the specimen (Fig. 6.1) and the groove weld was ground flush. This was done at the University of Alberta Structural Engineering Laboratory.

Since the stripping of the specimens was done by flame cutting rather than saw cutting, significant notches were created in the area of the weld in eight of the twelve specimens. In order to alleviate the problem of the notches, these specimens were milled back approximately 12 mm at each weld end, for a distance of 115 mm either side of the weld center (Fig. 6.2). (The flush-ground side of the weld detail can be seen in Figure 7.1). The other four specimens remained as initially cut.

Another problem encountered was shrinkage of the groove weld. The shrinkage was substantial and caused a bend in the specimens. Instead of the angle between the plates being 180° it was about 175° after completion of the welding. In order to test the specimens they had to be flattened. This was accomplished during the installation of the specimens in the testing apparatus.

6.3. Test Set-up

In order to subject the groove weld with backing bar detail to a fluctuating tensile stress, the following test set-up was used. Two C380 x 50 channels of CSA G40.21-M 300W steel (19) 4062 mm in length were placed back to back 128 mm apart. They were held stationary by a 13 mm x 300 mm x 1575 mm cover plate at the center on top, the 13 mm x 300 mm x 1400 mm groove weld specimen at the center on bottom, and by lacing placed as required. At either end of the specimen 13 mm x 300 mm x 838 mm splice plates were used to increase the section modulus of the test set-up for a further distance away from the constant moment region. The high-strength bolts used to fabricate this test set-up were A325M 22 mm diameter bolts 75 mm in length (Fig. 6.3). After two to three specimens had been tested, the channels had to be replaced because their useful fatigue life had been exceeded.

The span of the channels was 3658 mm and the weld detail was located at midspan, in the region of constant moment. Two point loads 662 mm apart were used to establish the constant moment region (Fig. 6.4). The channels were simply supported at the reaction points.

A Pegasus servo-load simulation system was used to provide the desired fluctuating load to the test specimen. The required force is produced by a controlled servovalve regulating the application of hydraulic power to an actuator. The maximum dynamic capacity of each jack is 489

kN. The two jacks were synchronized to provide the same force at the same time. The loading frequency was between 210 and 330 cycles/minute.

On the underside of the specimen three strain gauges were mounted 150 mm from the backing bar on either side of centerline. The strains were established from the selected stress range and minimum stress. The minimum and maximum loads were then established from the strains corresponding to the minimum and maximum stresses. The Pegasus controller was then used to apply and monitor the loads.

6.4. Test Procedure

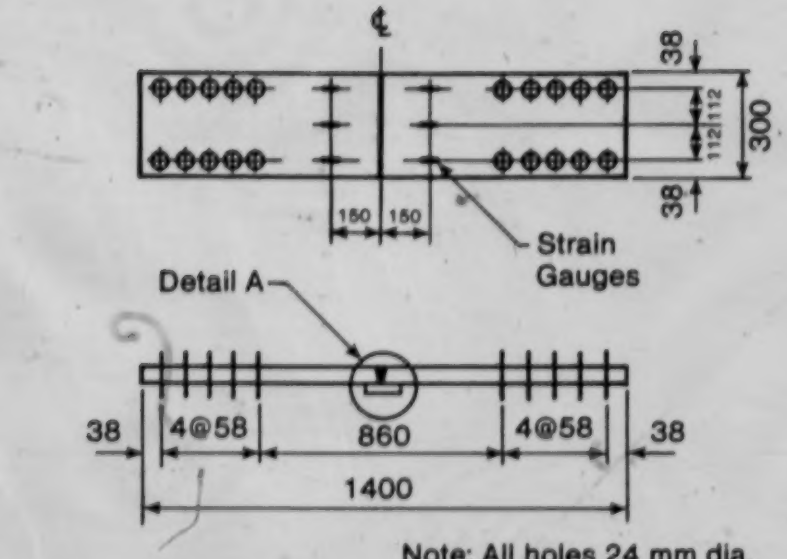
The first step was to install the groove weld with backing bar specimen in the test set-up. Since the specimen was slightly bent it was first loosely bolted in place and then four clamps were used to bring the specimen flush with the bottoms of the two channels (Fig. 6.5). After this was accomplished the bolts were snugged and then tightened by the turn-of-nut method (20). Strain gauge readings were taken at various stages during this process to obtain information about the stresses introduced. The same procedure was followed for all the specimens.

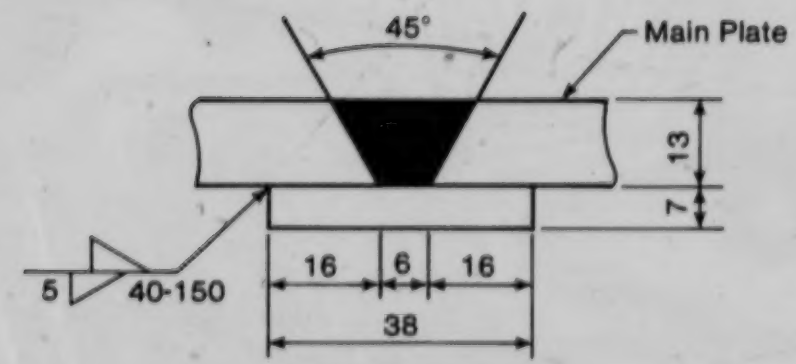
In order to acquire a good sampling of fatigue lives for different fatigue strengths the specimens were tested at various stress ranges between 80 and 120 MPa, with a minimum stress of between 20 and 35 MPa. The stress ranges were chosen on the basis of testing time, yield point of the

specimen, fatigue life of the channels, and capacity of the test set-up.

Although the specimens could not be visually inspected for fatigue cracks during the testing, failure was assumed to have occurred when the midspan deflection increased 0.5 mm (21,22).

Most of the time the fatigue tests ran continually twenty-four hours a day. After the fatigue testing was completed the crack was opened and the failure surfaces examined.





Detail A – Weld Profile

Figure 6.1 Details for Groove Weld with Backing Bar Specimens

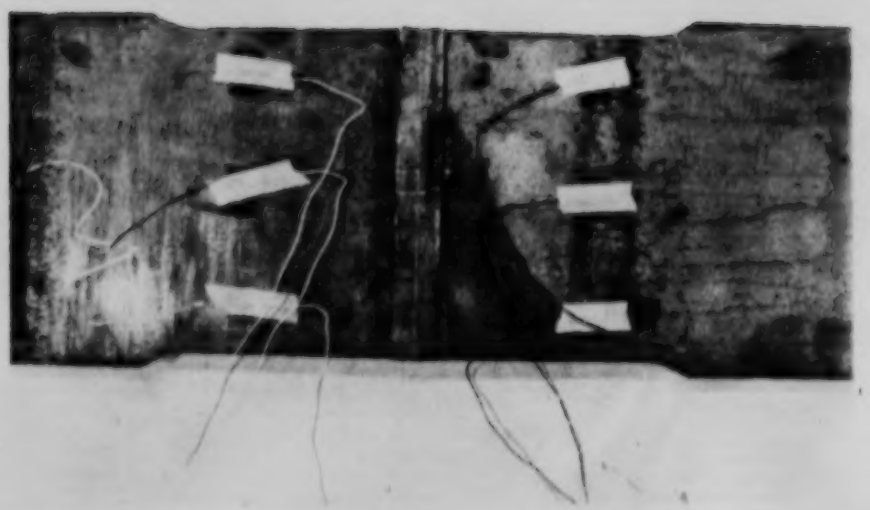


Figure 6.2 Typical Groove Weld with Backing Bar Specimen Before Testing

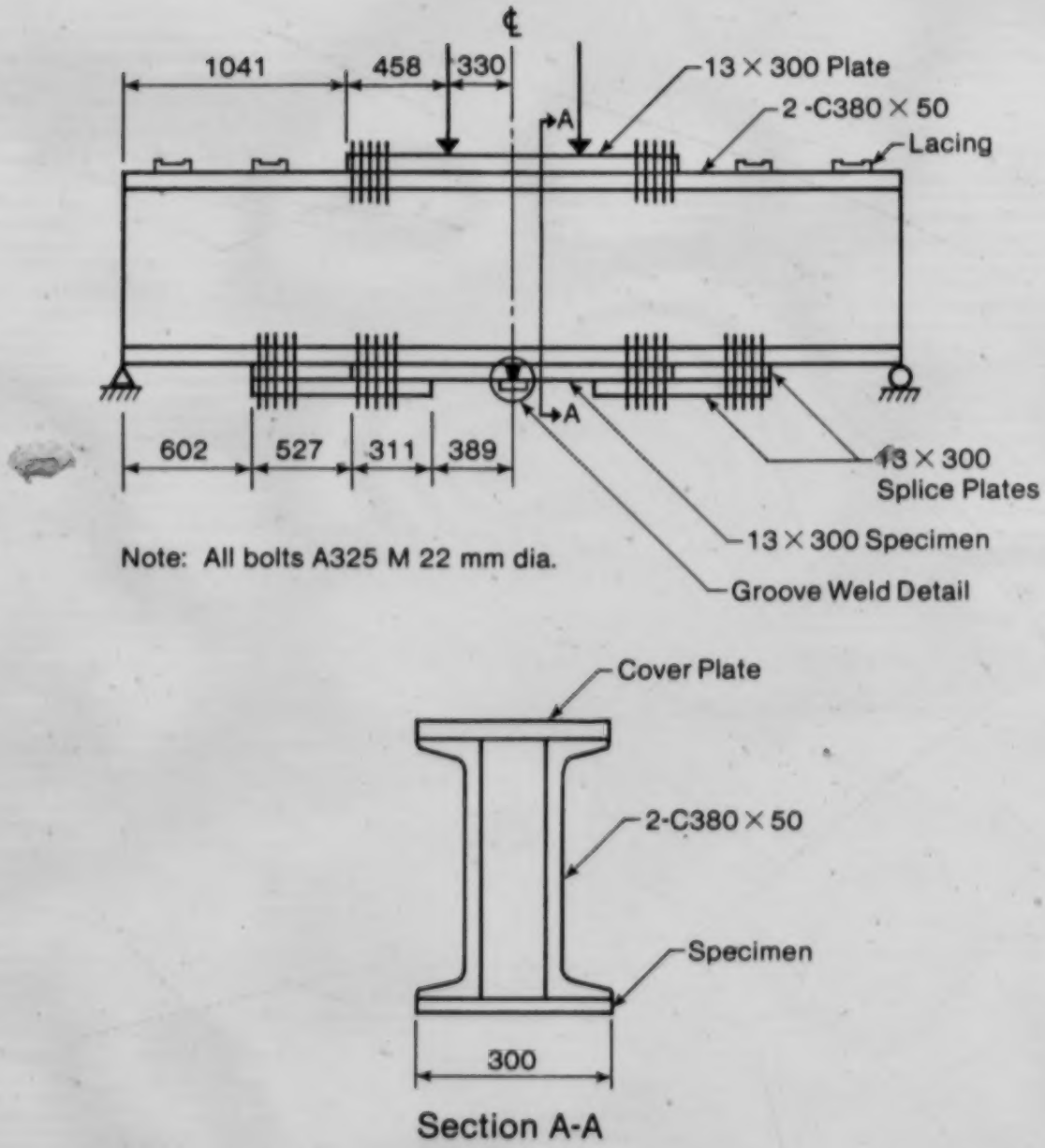


Figure 6.3 Test Set-up Details

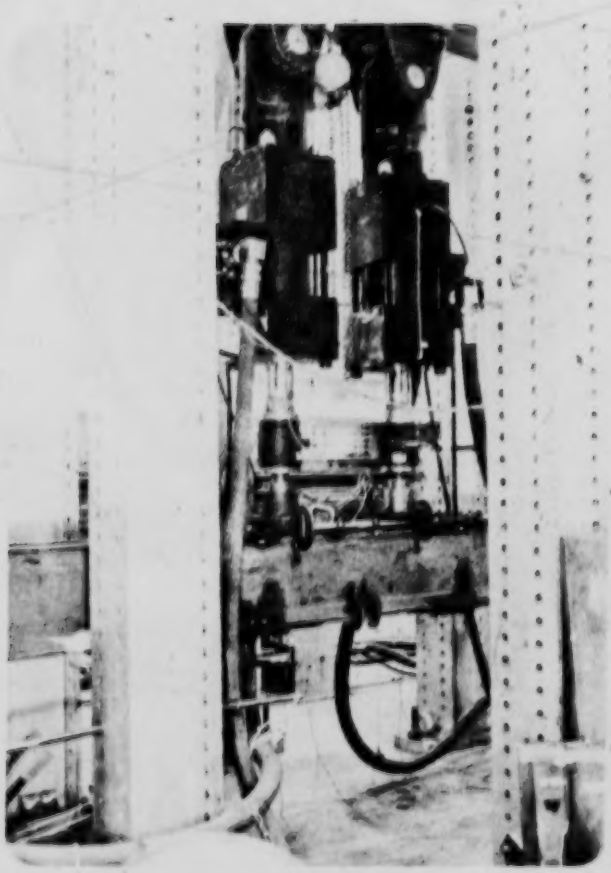


Figure 6.4 Test Set-up

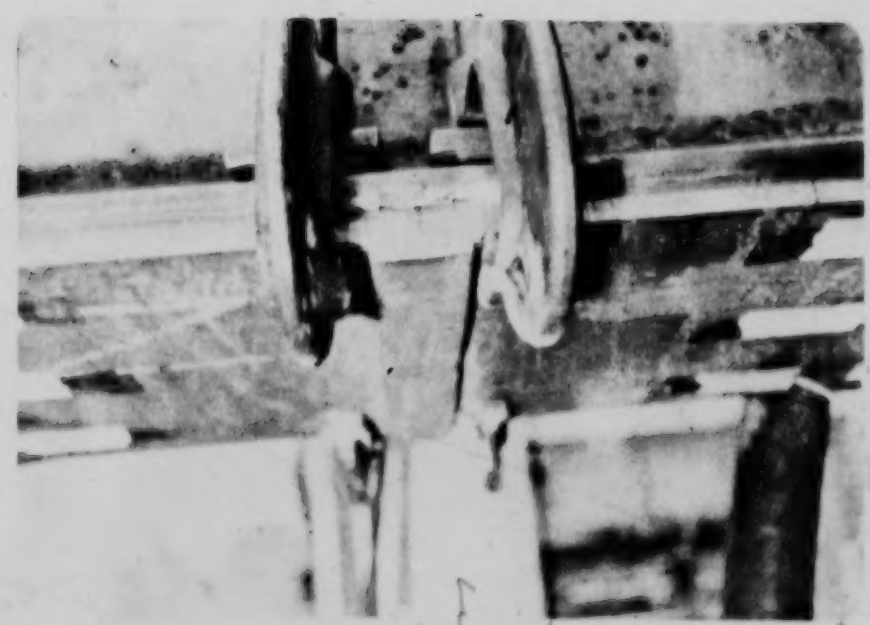


Figure 6.5 Clamped Specimen in Test Set-up

7. TEST RESULTS

7.1. Crack Initiation and Growth

In all of the groove weld with backing bar specimens the cracks initially began on the top surface, the same surface as the flush ground groove weld, in the center region of the plate width. This indicates that the center portion of each plate was probably a region of residual tensile stress. From the initiation site the cracks grew horizontally in both directions across the width of the plate and downward towards the backing bar. Upon examination of the failure surfaces it was apparent that the cracks started at a flaw and grew as a semi-ellipse. The cracks continued to grow semi-elliptically until the crack had propagated about 4 mm down through the thickness of the plate. At this time the cracks then grew as two-ended cracks until failure occurred. Failure was considered to have taken place when the dynamic deflection had increased by 0.5 mm.

In the three specimens tested at a stress range of 117 MPa all the cracks occurred along the edge of the groove weld in the heat-affected zone. At failure, the cracks had propagated to one of the edges of the plate (Fig. 7.1). The lengths of these cracks varied from 150 mm to 270 mm, or from 50% to 90% of the plate width. Two of these three specimens also had cracks throughout the length of one of the fillet welds used to attach the backing bar to the plate.

The specimen tested at a stress range of 110 MPa had a crack 128 mm in length, or 46% of the plate width. This crack extended along the edge of the groove weld in the heat-affected zone. Cracks also occurred through the complete length of one fillet weld and through one-half of another. These fillet welds were located on the same side of the backing bar.

Of the three specimens tested at a stress range of 100 MPa, two had a crack extending for 95% of the plate width. In one of these the crack went for 200 mm along the edge of the groove weld and for 100 mm in the groove weld itself. Also all the fillet welds located on one side of the backing bar had cracks throughout their length at failure. In the other the crack was entirely in the groove weld and no cracks were present through the fillet welds. The third specimen had a crack extending along the edge of the groove weld for only 16% of the plate width.

One specimen was tested at a stress range of 80 MPa. The test was allowed to continue for over six million cycles before it was stopped. The specimen never failed.

7.2. Finite Element Analysis

A finite element analysis of this detail was performed by Dr. Alaa Elwi. The joint considered was composed of two plates joined by a groove weld and a backing bar attached by 45° continuous fillet welds (Fig. 7.2). The analysis investigated the stress distribution of this detail. Since

one plane of symmetry exists, only one-half of the joint was considered.

In this analysis several assumptions were made. Firstly it was assumed that the backing bar and main plate were infinitely long, thus the problem was considered as a case of plane stress. Secondly, the welds and plates were considered to be of the same material. Thirdly, the material was assumed to be linearly elastic and isotropic. Fourthly, the fillet welds were considered to be continuous rather than intermittent. Fifthly, the effects of residual stresses due to welding were neglected.

In modelling the joint a quadratic serendipity element(eight nodes located at corners and midpoints) was used with 2 degrees of freedom associated with each node. The basic form of the mesh is shown in Figure 7.2. At the toe of the fillet weld, a region where a high stress concentration is expected, a 5 x 2 Gaussian rule was used to allow tracing of the rapid stress variation rate. In all the remaining elements a 2 x 2 rule was applied. Spring elements were used to simulate the boundary conditions.

An axial tension load of 100 MPa was applied uniformly to the extremity of the joint, that is, to the end of the main plate. This load was proportioned to the nodes to maintain a uniform distribution on the plate cross section.

Two variables were considered; the effect of the main plate thickness, and the effect of assuming the main plate and backing bar to be completely fused or fused only in the

weld regions. The main plate thickness was varied from 13 mm to 29 mm while the backing bar thickness remained constant at 7 mm. This was modelled by varying the locations of the nodes only in the Z direction. In all, six different cases were considered.

The computer program used to carry out the analysis was FEPARCS5 (32). Seven computer runs were done, six assuming that the backing plate and the main plate were completely fused while varying the main plate thickness, and one assuming that the plates were fused only in the weld regions.

The distributions of stress were analysed for the elements at three locations: through the thickness of the main plate, on the surface of the stressed plate adjacent to the fillet weld toe, and on the plane perpendicular to the plate surface through the center of the groove weld. The output provided the stresses at each Gaussian point in every element. This included the major and minor principal stresses in the R-Z plane as well as the normal and shear stresses in the R and Z directions.

At the edge where loads were applied, across the thickness of the main plate, the stress distribution was uniform to within 1%.

The output shows that the highest stress concentrations are located at the fillet weld toe. They are, however, very localized. The observed magnitude of stress at the toe line, on the main plate, was 184 MPa and this did not vary with

changes in the plate thickness. The stress concentrations in the weld itself at the toe were even higher. They varied from 186 MPa for a main plate thickness of 13 mm to 252 MPa for a main plate thickness of 29 mm. These major principal stresses described angles, relative to the R axis, of 86° and 87°, respectively (Fig. 7.3). It appears these stresses tend to separate the weld from the main plate. The stress adjacent to the fillet weld toe decreases rapidly with distance from the toe. The results indicate that the influence of the stress concentration virtually ceases to exist at a distance approximately equal to 0.5 times the plate thickness away from the centerline of the groove weld. (In Gurney's study (26), where the joint analysed consisted of two attachments fillet welded to a main plate, this distance was found to be 0.625 times the main plate thickness.) Since a sharp corner was used in modelling the fillet weld toe the stress concentration at this point would theoretically be infinite. In reality the stresses here would depend upon the radius of the fillet weld. If a more favorable weld profile was used these stresses would be expected to be ameliorated.

The stress distribution along a centerline through the backing bar was eccentric. A steep gradient existed with high stresses at the face of the groove weld. These stresses varied with the main plate thickness. For a main plate thickness of 13 mm the stress varied from 9 MPa at the top of the backing bar to 128 MPa at the face of the groove

weld. When the main plate thickness was increased to 29 mm the stresses increased from 34 MPa at the top of the backing bar to 120 MPa at the groove weld surface (Fig. 7.4). As the main plate thickness increases, the stress concentration at the face of the groove weld decreases and the stress concentration at the backing bar increases.

Whether or not the backing bar and the main plate were considered to be completely fused or only fused in the weld regions, it was found that there was no difference in the results for the same plate thickness. Therefore, within the width of the backing bar and weld the joint acts as a complete assembly with no separation surface.

7.3. Comparison of the Experimental Results and the Finite Analysis

The actual crack initiation sites were compared to the highest stress concentration locations predicted by the finite element analysis. In the tested specimens the cracks began on the same side as the groove weld face and propagated outwards and downwards. Although the finite analysis showed that high stresses occur at this location, they are not the highest in the cross section. The highest stresses occur at the fillet weld toe. Whenever behavior is described by an analytical model it must be kept in mind that the model can only approximate the actual behaviour. The use of discrete elements to describe the behavior of a complex state of stress reliably depends on how well the

elements represent true displacement fields and how well the actual loading conditions are simulated. Here the use of a sharp corner for the weld profile is probably not realistic. In reality the weld profile would usually be curved to some degree. The use of a rounded corner would reduce the stress concentration at this point. If one accepts that the fillet weld profile is less severe in the experimental prototype than in the analytical model, then the finite element analysis is a good predictor of stress distribution.

7.4. Comparison with Specifications

The results of the tests on groove welds with backing bars are plotted in Figure 7.5 and can be compared with Category C of the CSA Standard (20). The data points all fall to the right of this category.

As has been noted, current CSA and AISC (18) specifications do not permit the backing bar to remain in place, presumably, because of the inevitable high stress concentrations that will occur at the fillet welds. If the backing bar is removed, the detail that would remain would be a flush ground groove weld which these standards would designate as Category B. Category B is also shown in Figure 7.5. The experimental results obtained from this investigation show that leaving the backing bar in place does not result in a drastic reduction in the fatigue life as compared to the case of a flush ground groove weld. In Figure 7.5 the results are compared with Category B and C.

The results fall to the left of Category B and to the right of Category C. Therefore, the suitable design choice is Category C. This detail is certainly not one of the details with particularly low fatigue life.

The British Standard BS 5400, Part 10 states that the fatigue strength of this detail would fall into Class F or Class G, depending on the location of the fillet welds relative to the member edge. Class G applies when the backing bar is permanently fillet welded within 10 mm of the member edge. This was the situation with all the members tested herein. It was not within the scope of this study to investigate the effect of lateral placement of the fillet weld on fatigue life. This investigation has shown that the life of a groove weld with backing bar attached by intermittent fillet welds, where the stress is applied perpendicularly to the detail, is much greater than either Category F or G.

7.5. Comparison with Previous Studies

In the literature survey, several experimental fatigue stresses for a groove weld with the backing bar attached at the root of the groove weld were noted. For 2×10^6 cycles the stress range varied from 100 to 169 N/mm^2. From the mean regression line established from the statistical analysis of this test series at 2×10^6 cycles, the stress range would be 111 N/mm^2. The presence of the fillet welds does not seem to have a large effect.

In the East German study (30) the fatigue lives of the detail of a groove weld with a backing bar attached in three different ways was investigated. The backing bars were attached by fillet welds on both sides, on one side, or at the root of the groove weld. It was concluded that there was no significant difference in the fatigue lives of the three types, and no preference could be given as to the best way to attach a backing bar. From this test series the stress range varied from 104 to 144 N/mm^2 at 2×10^6 cycles. This is consistent with the results obtained from the investigation reported herein. In the case where the backing bars were attached by fillet welds on both sides cracking started at the top, that is, at the same side as the face of the groove weld, and proceeded through the basic material. The backing bar actually provided protection.

7.6. Effect of Stress Range

One of the principle factors affecting the fatigue strength of the groove weld with backing bar detail was the stress range. Throughout the test series the minimum stress was kept between 20 MPa and 35 MPa with the stress range being varied. The fatigue life of the detail decreased when the stress range was increased, as was expected.

The test data for the groove weld with backing bar specimens tested are presented in Table 7.1 and are plotted in Figure 7.5. Each data point on the graph represents a separate fatigue crack.

A linear regression analysis was carried out on the test data using the model $\log N = A + B \log S_r$. This line, $\log N = 14.822 - 4.163 \log S_r$ is plotted in Fig. 7.6.

This mean regression line has a negative slope of 4.2. When the British Standard BS 5400, Part 10 (28) was being compiled it was noted that of six details involving transverse welds five of them had slopes greater than 2.92 and four of them were in the range 3.47 to 3.69 (29). In contrast to this seven of the nine details involving longitudinal welds had slopes from 2.17 to 3.08. The details where transverse welds were present had shallower stress range versus number of cycles lines. This was attributed to the differences in residual stresses.

7.7. Effect of Straightening the Main Plate

The small bend in the plates was the result of weld shrinkage. The straightening of the plate, accomplished by the use of clamps, inevitably introduced a residual stress system. However, it was considered that the stress field formed as these laboratory specimens were straightened would be similar to that which would occur in the field as initially flat plates were welded and not permitted to distort. From the strain gauge readings taken before and after the main plate was straightened a definite change occurred. The magnitude of the change varied considerably from specimen to specimen. In situations where the joint is subjected to tensile loadings only, the residual stresses do

not influence the fatigue strength. The residual stresses that result from welding equal the yield stress of the parent material at some points and the actual stress cycle the material is subjected to will vary from the yield stress downwards regardless of the nominal stress cycle (29).

The assumption that the fatigue strength of this detail, a groove weld with backing bar subjected to a perpendicular stress, is not affected by straightening the main plate is supported by the results of the East German study (30).

Table 7.1 Summary of Results for Groove Welds with Backing Bar Specimens

Specimen No.	Stress Range (MPa)	Failure (cycles x10^3)	Crack Initiation
3	100	4485.5	TP
6	117	1978.8	TP
7	117	1196.6	TP
8	80	6216.9	—
9	117	1290.3	TP
10	100	4128.8	TP
12	100	2991.4	TP
2	110	2113.2	TP

In all cases, σ_{min} was between 14 and 21 MPa

TP=top of plate, side of flush ground groove weld

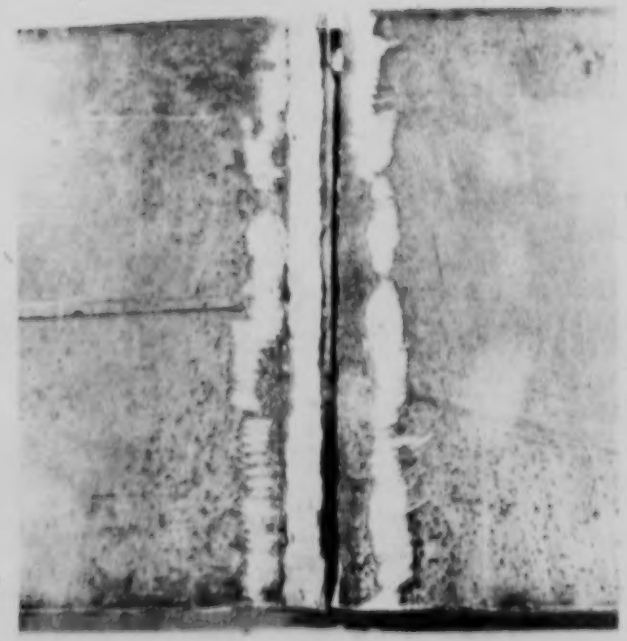


Figure 7.1 Failure Crack in Groove Weld with Backing Bar Specimen

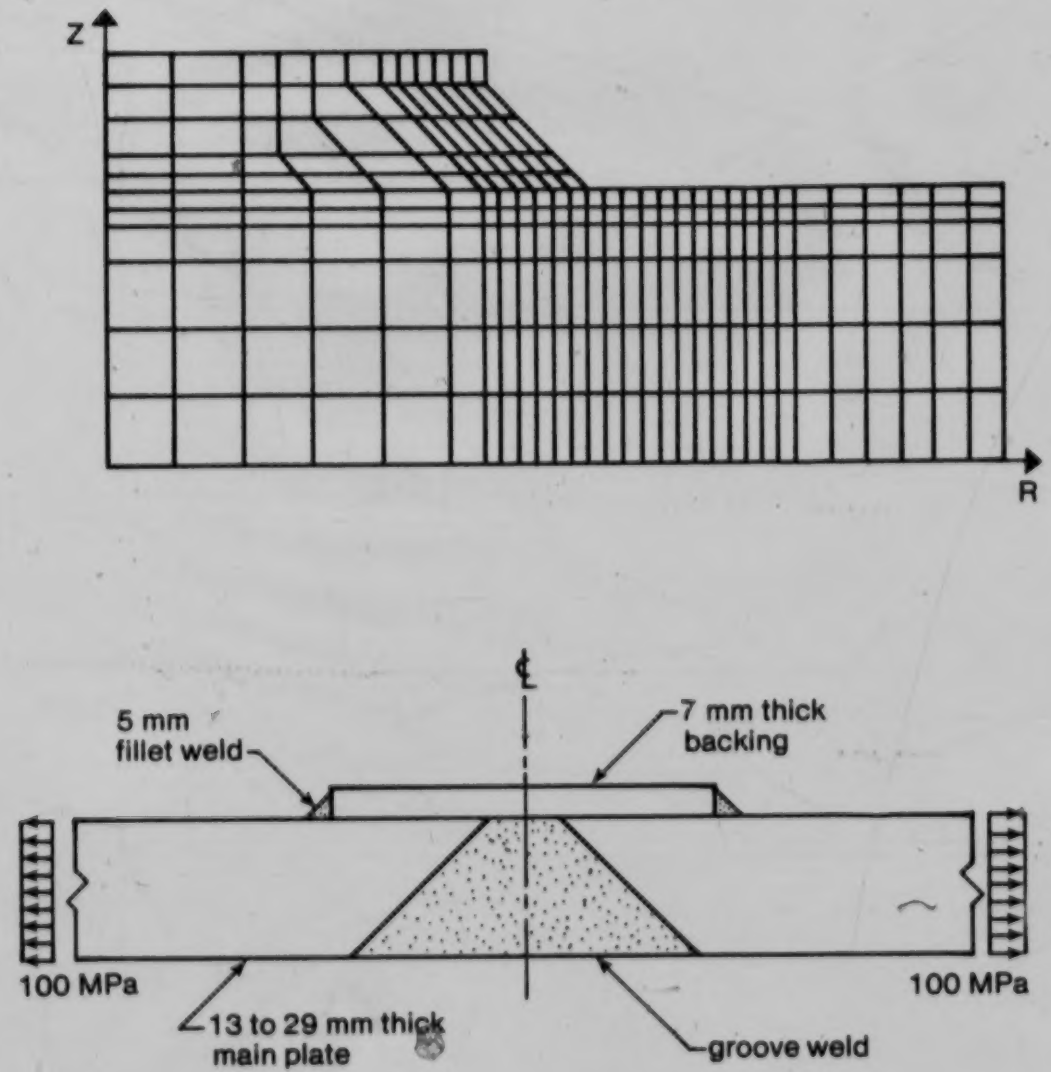
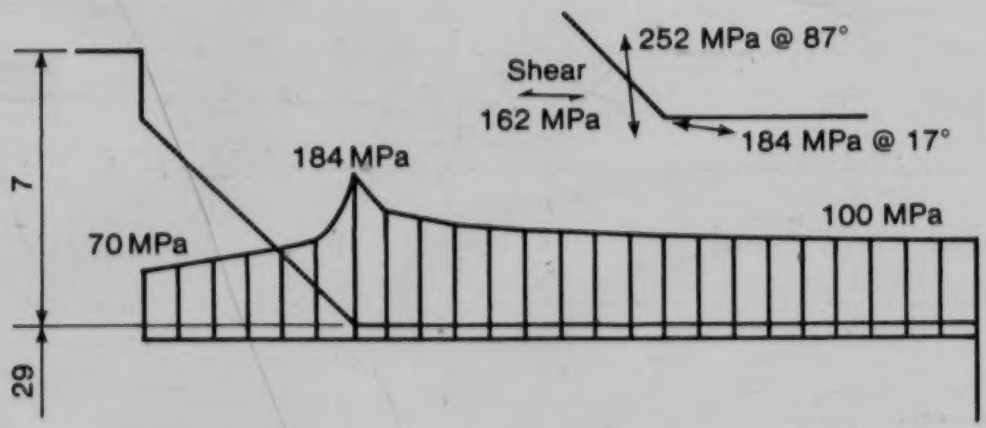


Figure 7.2 Finite Element Model and Mesh of the Groove Weld with Backing Bar



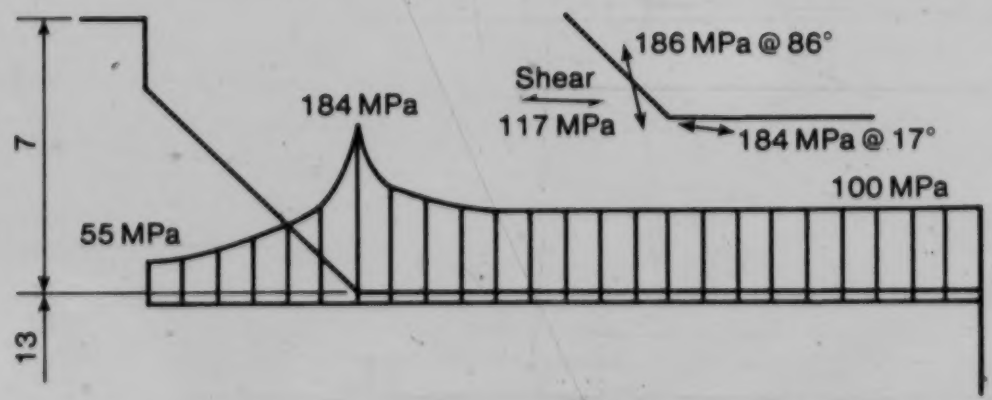


Figure 7.3 Regions of High Stress for the Groove Weld with Backing Bar

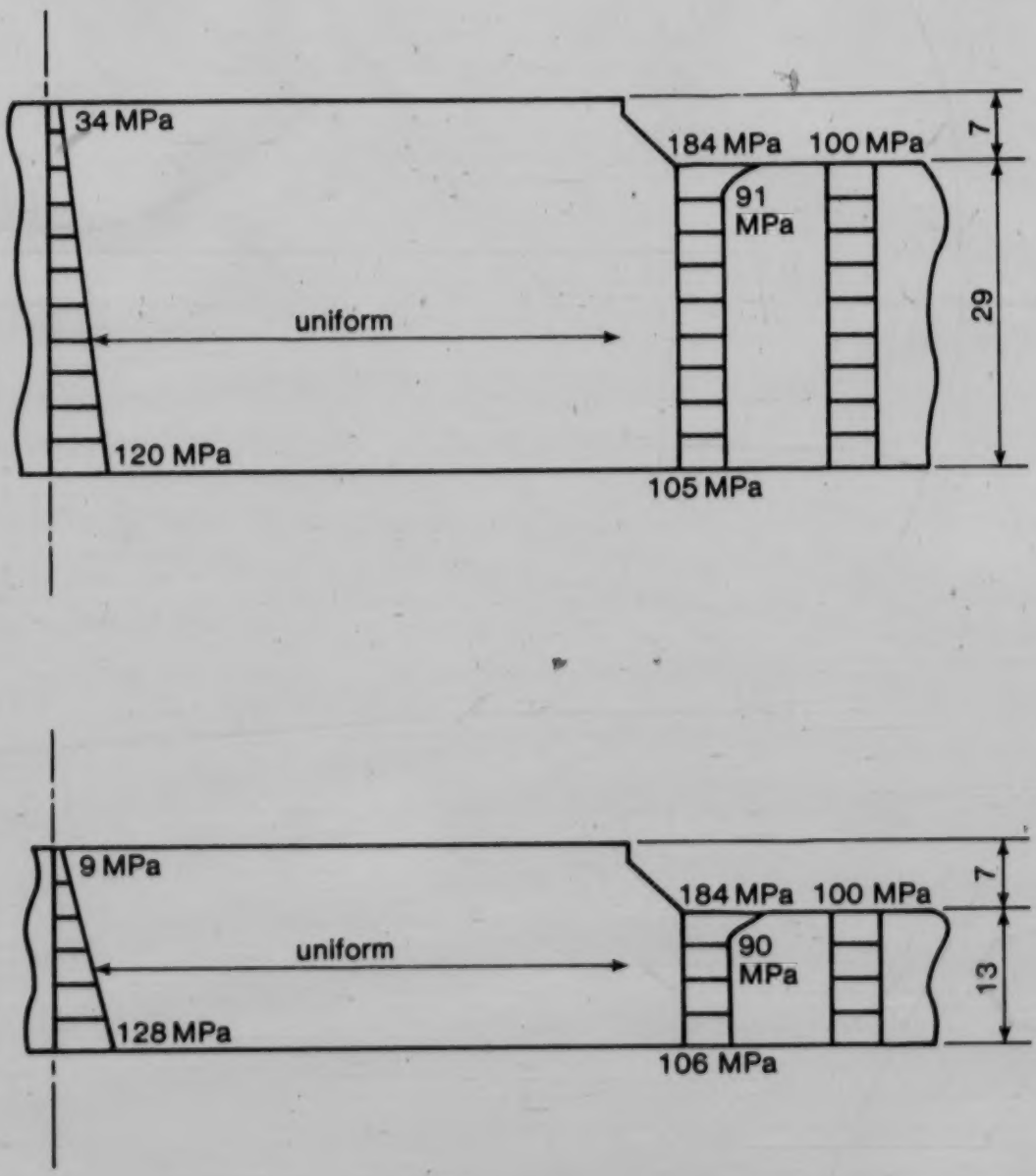


Figure 7.4 Regions of High Stress for the Groove Weld with Backing Bar

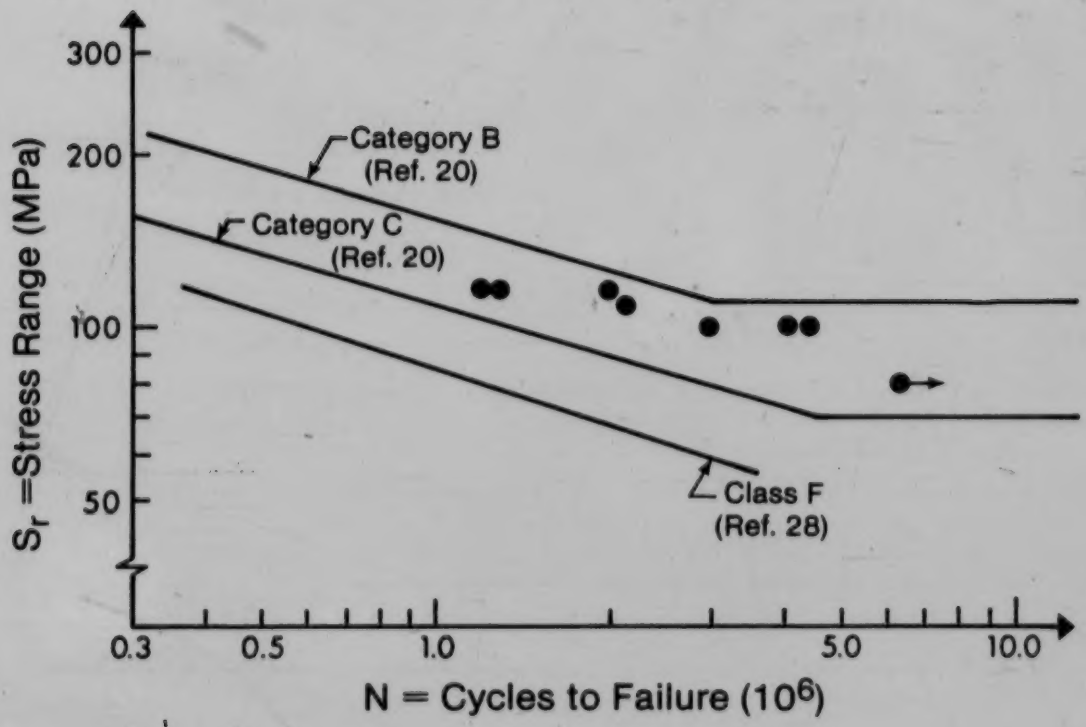


Figure 7.5 Groove Weld with Backing Bar-Test Results

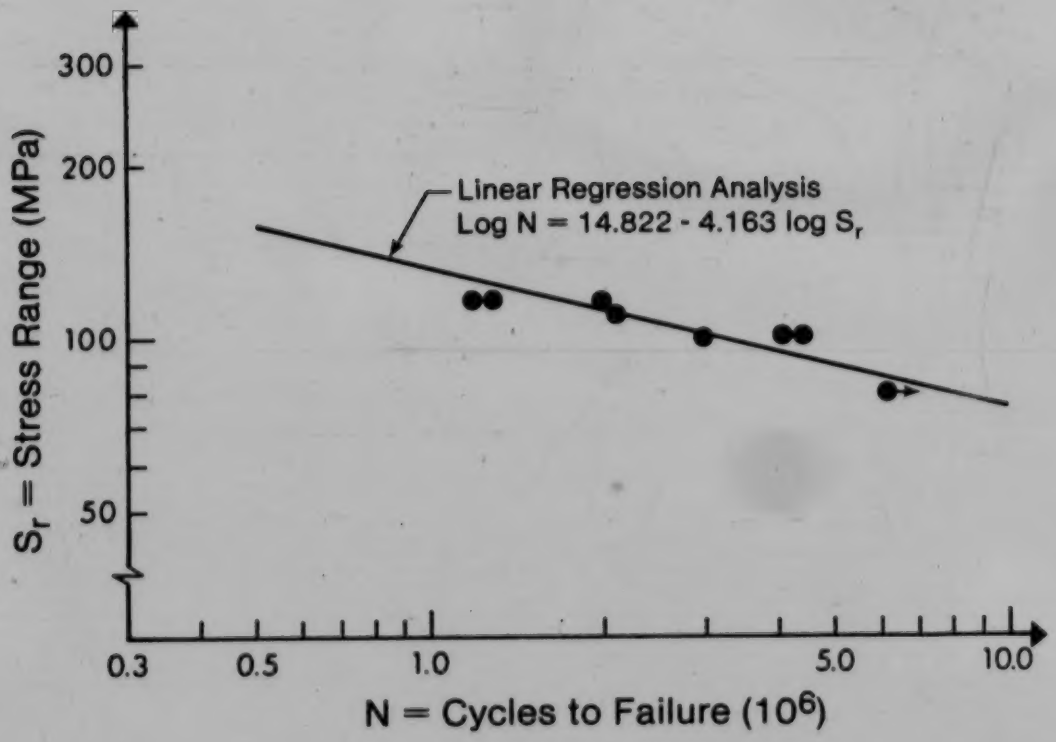


Figure 7.6 Groove Weld with Backing Bar-Test Results

8. SUMMARY and CONCLUSIONS

8.1. Summary

In Part A of this investigation, three details were examined to study the fatigue life of riveted connections. Beams with empty holes were studied to establish a reasonable expectation for the fatigue life of a connection with loose rivets, beams with high-strength bolts in the holes were investigated to estimate the benefits to be gained by extending the fatigue life of riveted members by replacing rivets with bolts, and actual riveted bridge members taken from service were examined.

The experimental program consisted of six tests on wide flange beams, three with holes and three with a bolted detail, and four on actual riveted members. The latter gave eleven test results. The failure location, the effect of bolting, the effect of the clamping force, and the testing procedure were considered. For the actual riveted members the effect of the previous stress history was taken into account. The information obtained from this investigation was compared with other investigations and the recommendations of current design specifications.

In Part B of this investigation the fatigue behavior of groove welds with backing bars located transversely to the direction of applied stress was studied. The backing bars were attached with discontinuous fillet welds. This detail was examined experimentally and analysed theoretically.

The experimental program consisted of eight tests. The main parameter considered was the effect of the stress range on the fatigue life of the detail. The location of the initial crack causing failure was also studied. The information gathered from the testing was compared with the recommendations of current design standards.

The theoretical program consisted of a finite element analysis. Two variables were considered; the effect of varying the main plate thickness, and the effect of considering the backing bar and the main plate to be completely fused or only fused in the weld regions. Seven computer runs were made. Stress distributions for the weld detail were obtained. Crack locations observed in the test program were compared with locations of high stress given by the finite element analysis.

8.2. Conclusions and Recommendations

Part A - Riveted Connections

The investigation has produced the following findings:

1. The detail of beams with holes simulates either a loose riveted connection (but one in which any stress concentration from rivet bearing is not significant) or simply a location where a hole has been left unfilled. The observed fatigue lives for this case were less than those predicted by the AASHTO Specification for a riveted connection. All beams with holes failed due to a crack at the extremity of the first hole in the line of

stress.

2. The detail in which holes were filled with high-strength bolts produced fatigue lives that were significantly greater than those predicted by current specifications for bolted connections. They approached the predicted fatigue lives for plain material. In two cases failure occurred as a result of a crack at the extremity of the first hole in the line of stress and in one case due to a crack at the extremity of an interior hole.
3. In all cases the rivet details taken from actual bridge members produced fatigue lives greater than those predicted by the AASHTO Specification for a riveted connection. The bridge specimens failed due to cracks at the rivet holes, except for one case where failure was due to a crack initiated at a flaw produced by grinding.
4. The scatter in the results of the riveted specimens appears to be caused by differences in the clamping forces.
5. The performance of beams with bolts, which had fatigue lives fourteen to twenty times greater than the beams with empty holes, reflects the benefits that can be gained from replacing rivets with high-strength bolts.
6. Based on these test results, it can be concluded that improperly located holes in a structural member can be filled with bolts to produce a fatigue life that will be near that predicted by North American specifications for plain material.

7. In structural members with riveted connections Category D of the AASHTO Specification should continue to be used. In cases where the rivets are suspected of being loose this category does not provide a safe prediction of the fatigue life. Category E should be used for predicting the fatigue life of this type of detail, a loose riveted connection.
8. In designing bolted connections for fatigue, Category B of the current specifications should continue to be used.
9. Fatigue life of a riveted member can be extended by replacing the rivets with high-strength bolts.

Part B - Groove Welds With Backing Bars

The findings of this investigation are:

1. The detail of a groove weld with backing bar located perpendicularly to the direction of applied stress, where it is attached with discontinuous fillet welds, had a fatigue life much greater than that predicted by the British Standard BS 5400, Part 10. North American specifications currently do not permit this detail to be used at all.
2. In all cases the failure crack began on the same surface as the face of the groove weld. In four cases the crack ran parallel and along the edge of the groove weld in the heat-affected zone. In the other cases the crack was

present not only along the edge of the groove weld in the heat affected zone, but in some parts in the groove weld material. Cracks all started on the surface and propagated downwards and outwards.

3. The fatigue life decreased as the stress range increased, as expected.
4. The mean regression line for this detail had a negative slope of approximately 4.0. The lines obtained in earlier studies on other details (1,22) had a negative slope of about 3.0.
5. The finite element analysis showed the highest stress concentrations to be at the toe of the fillet weld. However, this analysis assumed a sharp corner for the fillet weld. In reality this profile is likely to be rounded and thus the stress concentration would be reduced. No variations in the stresses occurred with changes in the main plate thickness.
6. From the finite element analysis high stress concentrations were also shown to occur at the surface of the groove weld. The stress distribution through the thickness of the plate and and backing bar decreased considerably as the surface of the backing bar was approached. These stresses varied with the main plate thickness decreasing at the groove weld surface and increasing at the backing bar surface as the main plate thickness increased.
7. In comparing the experimental results and the finite

element analysis the location where failure occurred in the tested specimens coincided with a location of high stress defined by the finite element analysis.

8. Whether the backing bar and main plate were fused completely or fused only in the weld regions had no effect on the results. In both cases no separation surface existed.
9. When this detail is present in a structure and subjected to fatigue loading, Category C of the AASHTO or CSA specifications should predict the fatigue life safely.
10. If further testing was undertaken the test set-up should be redesigned. In the initial planning the sizes of channels chosen were considered adequate to allow testing at a wide range of stress ranges, however, it was found that the detail had a much greater fatigue life then was anticipated. Therefore, the upper stress ranges could not be developed.

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